Appendix G

Conceptual Designs for High Priority Sites

Concept Design OO_04B

1 EXISTING SITE DESCRIPTION

The questionnaire responses indicate that water ponds on River Road approximately 2,000 feet west of Chief Road and onto adjacent properties several times a year. Figure 1 and the photographs in this document show the existing site conditions.

There is a 1.5-foot-diameter high-density polyethylene (HDPE) culvert at this location that drains a 1.5-square-mile watershed. The existing culvert appears to be located under or directly adjacent to the residence of 32006 River Road and a shed on the 32018 River Road property.

During the first field investigation (September 2014), ponding water was observed on the roadway, and collapsed concrete was covering the culvert crossing River Road. Tidal water was also observed flowing upstream from the culvert. A catch basin connected to the culvert had standing water. During the second field investigation (February 2015), damage to the culvert downstream of River Road was observed.

The minimum elevation of River Road is approximately 2 feet North American Vertical Datum 1988 (NAVD88). Under current conditions, runoff from the watershed drains slowly through the existing culvert and often floods River Road. Tidal water exacerbates this problem by raising water levels in the marsh to the north.

Collapsed headwall over the existing River Road culvert approximately 2,000 feet west of Chief Road

Damaged culvert at the Indian River Bay downstream of River Road

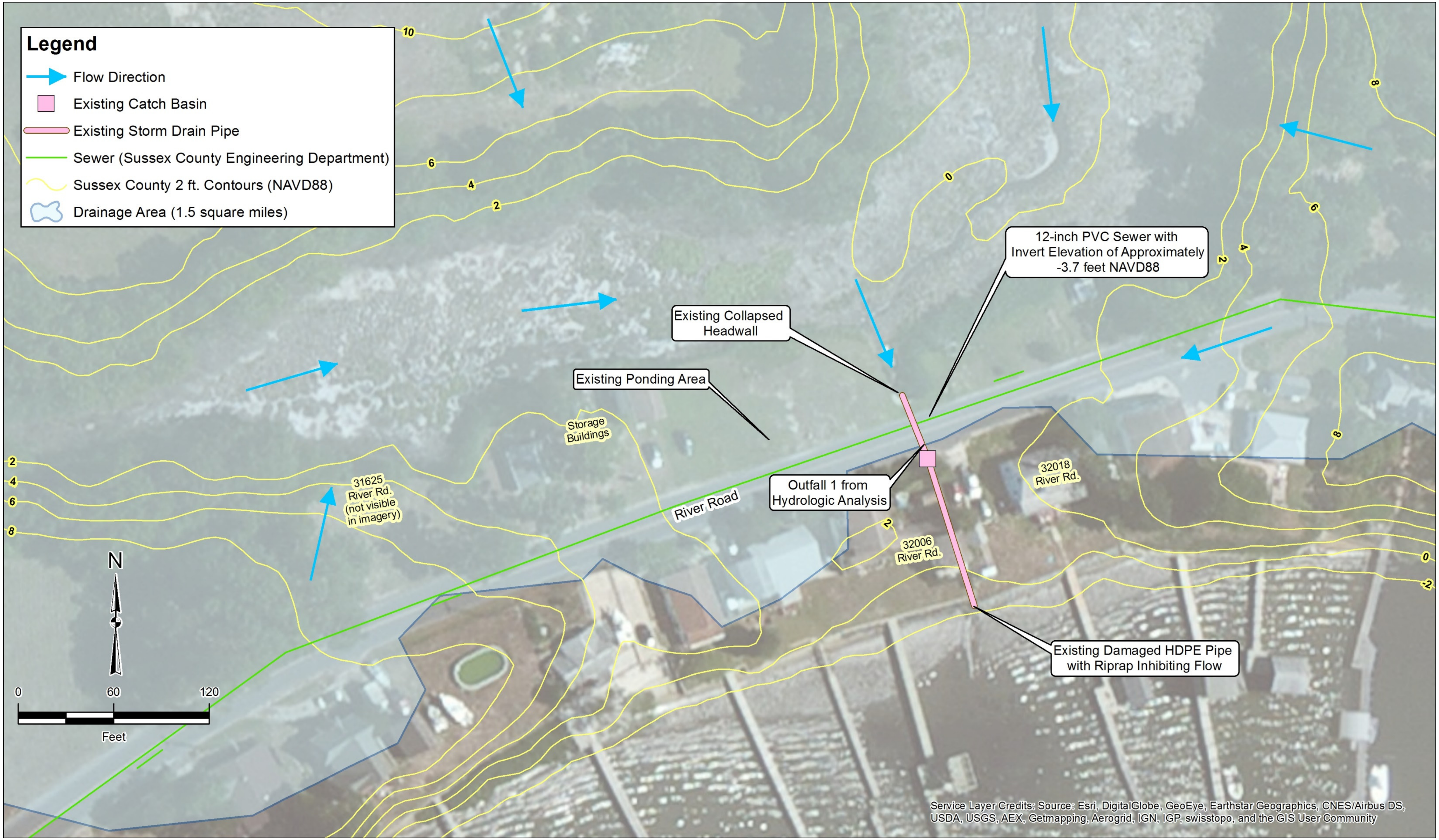

Figure 1: OO_04 Existing Site Conditions

2 PROPOSED IMPROVEMENT

The proposed design at site OO_04 is to create additional storage by installing approximately 1,000 feet of bulkhead, and to increase conveyance by installing three 30-inch diameter culverts under River Road and installing three catch basins. Figure 2 displays the proposed system. This system would be sized to convey the 25-year storm event (in accordance with the Delaware Department of Transportation Road Design Manual [DelDOT, 2008]). Presently, River Road controls storage within the marsh, and it floods yearly along with nearby properties.

Collapsed headwall over existing River Road culvert approximately 2,000 feet west of the Chief Road during high tide

The proposed bulkhead should be composed of impervious material (e.g., concrete). The proposed headwall will be at an elevation of 5.25 feet NAVD88, while the remainder of the proposed bulkhead would be at an elevation of 5.5 feet NAVD88 (approximately 2.5 feet above the ground elevation at that location). This would increase storage without negatively affecting residences upstream or downstream. Any overflow would be concentrated to River Road because of the lower headwall, and would then spread over the road and low lying areas until marsh levels or tide levels were low enough to allow the catch basins to drain the ponded water. This would prevent adverse effects to the two storage buildings and the residence of 31525 River Road located between the proposed bulkhead and River Road. 31525 River Road is the only residential structure between the proposed bulkhead and River Road, and it has a first floor elevation of approximately 7 feet NAVD88, so negative impacts are not anticipated. The bulkhead would be built with over 1 foot of freeboard above the design storm elevation. Alternatively, the bulkhead and headwall could be installed at a lower elevation if the conveyance of the system is increased or if a lower design storm is used.

Due to the proximity of the existing culvert to adjacent properties, it is recommended that the proposed culverts be installed under the driveway of 32018 River Road. At the current elevation of River Road, it appears that a 30-inch diameter pipe is the largest that could pass under the road, although a detailed survey will be required for verification. Three 30-inch diameter culverts would be required to convey the 25-year storm event. A headwall is recommended both upstream and downstream of the culvert. Backwater control check valves (e.g., Tideflex CheckMate inline check valve) are recommended at the proposed junction box or at the downstream end of the proposed culverts.

To reduce nuisance flooding on River Road, three catch basins are proposed at low areas in the vicinity of the existing culvert crossing River Road. Storm drain pipe would connect the northern catch basin to the marsh, and the two southern catch basins would connect to the proposed junction box. The minimum DelDOT recommended storm drain pipe size of 15 inches is proposed. Backwater control check valves are recommended in the storm drain pipe to prevent surcharging into River Road when marsh levels are elevated. The inline check valves could be located at either the upstream or downstream end of the pipes, depending on preferred maintenance locations.

Approximate location of existing culvert between 32006 River Road (left) and 32018 River Road (right)

Based on site investigations, it appears that the existing culvert crossing River Road has adequate capacity to convey small flows (less than the 2-year storm event), and therefore may be left in place with repairs to the upstream and downstream segments. Further investigation of the pipe is required, and slip lining is recommended if the pipe is in poor condition. Removing and replacing the existing culvert would not be viable given the proximity of the pipe to residential structures. A headwall and tide gate (e.g., the Waterman self-regulating tide gate) is recommended downstream of the existing culvert to allow saltwater to flow into the marsh during low and average tides while preventing flow during high tide. By installing backwater control check valves for the proposed culverts and a tide gate for the existing culvert, the maximum flow rate out of the marsh would be higher than the flow rate into the marsh. It is recommended that the existing catch basin be converted to a junction box to prevent surcharging due to marsh backwater.

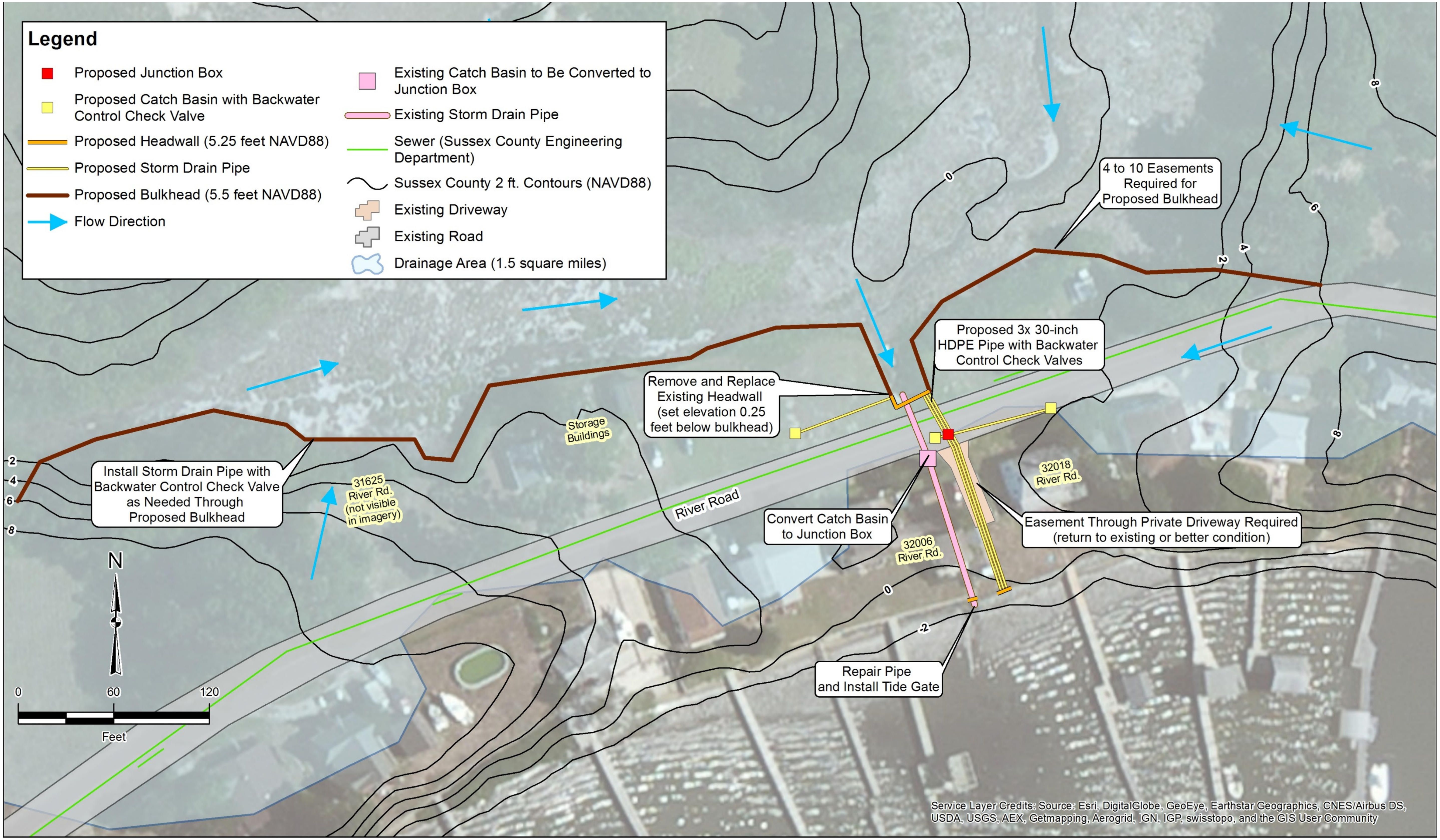

Figure 2: OO_04 Proposed Site Design

3 HYDROLOGIC CALCULATIONS

3.1 HEC-HMS Hydrologic Analysis

A hydrologic analysis for Oak Orchard was performed using HEC-HMS. The methodology and results of this study are discussed in Appendix F. The peak flows into the existing culvert at OO_04 are calculated at Outfall 1 (Figure 1). These flows do not account for the flow through the existing culvert or the storage behind the culvert, as these factors will vary with the proposed design. The peak flows to the existing culvert for the 2-, 10- , 25-, 50-, and 100- year-annual-chance recurrences without accounting for culvert hydraulics or storage are approximately 80, 170, 250, 330, and 410 cubic feet per second, respectively. Figure 3 displays the inflow hydrographs calculated in HEC-HMS. These inflow hydrographs are used with storage information to compute hydraulic discharges and calculate the peak flow through the existing and proposed culverts. Due to storage upstream of the existing and proposed culverts, the peak flow out of the culverts must be less than or equal to the peak of the inflow hydrographs.

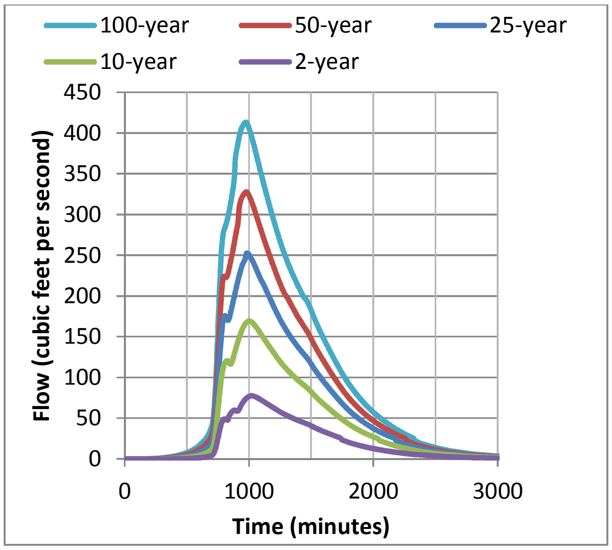

Figure 3: OO_04 Inflow Hydrographs Calculated in HEC-HMS

3.2 Storage Calculations

The marsh upstream of the existing and proposed culverts at OO_04 provides storage for the flow upstream of the existing and proposed culverts crossing River Road. The National Elevation Dataset (NED) 1/9-arc-second (3-meter) raster from the U.S. Geological Survey (USGS) was used to calculate the storage upstream of the existing and proposed culverts (USGS, 2007). The vertical datum for the topographic data is NAVD88. The high point for the proposed design is the proposed bulkhead elevation (minimum elevation of 5.25 feet NAVD88), while River Road is the high point for the existing conditions (the minimum road elevation is approximately 2 feet NAVD88).

The elevation raster was converted to a triangulated irregular network over the storage area, and the polygon volume GIS function was used to calculate the volume for various elevations (ESRI, 2012). It was assumed that under existing conditions the marsh level is most likely to be at the average tide elevation (0.5 foot NAVD88), while the proposed marsh elevation is assumed to be the average low tide elevation (-0.83 foot NAVD88) due to the backwater control check valves and the tide gate. A detailed discussion of the tidal elevations is included in Section 4.1.

Table 1 shows the storage volume for the existing and proposed site conditions, and Figure 4 displays the proposed bulkhead and upstream area with 2-foot contours.

Table 1: OO_04 Existing and Proposed Conditions Stage-Storage Relationship

Elevation (feet NAVD88)	Existing Conditions Storage Volume (acre-feet)	Proposed Conditions Storage Volume (acre-feet)
-0.83	0.00	0.0
-0.5	0.00	0.3
0	0.00	1.0
0.5	0.00	3.4
1	4.94	8.2
1.5	11.35	14.4
2	18.49	21.2
2.5	26.24	28.6
3	34.55	36.4
3.5	43.31	44.7
4	NA[a]	53.4
4.5	NA[a]	62.6
5	NA[a]	72.2
5.5	NA[a]	82.3
6	NA[a]	92.9
6.1	NA[a]	95.1

[a] Not applicable because the road begins to be overtopped at an elevation of 2 feet NAVD88, and elevations are not anticipated to exceed 3.4 feet NAVD88

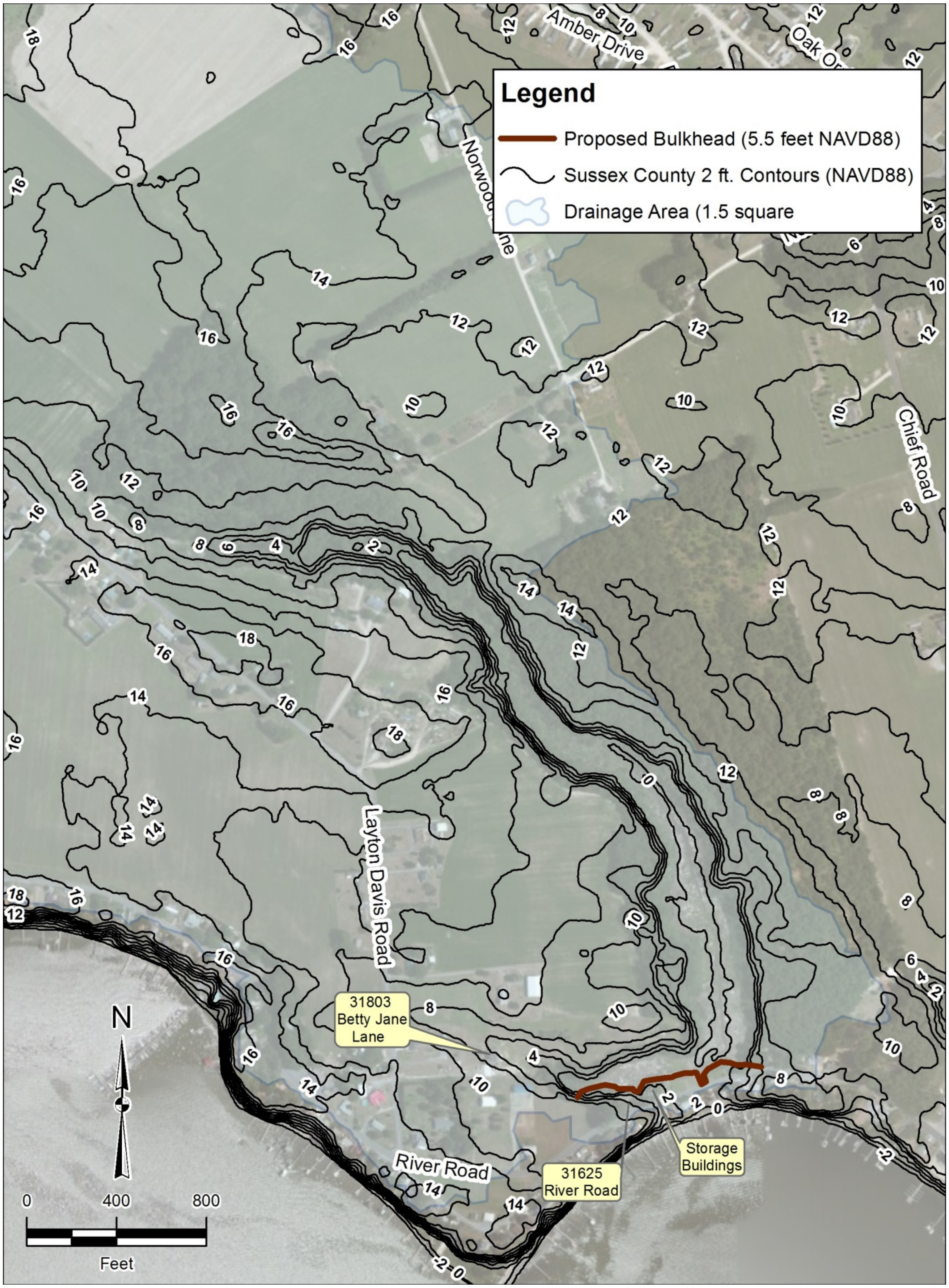

Figure 4: OO_04 Proposed Bulkhead and Upstream Storage Area

4 HYDRAULIC CALCULATIONS

4.1 Tidal Tailwater Boundary Conditions

The hydraulic conditions of this site are influenced by the Indian River Bay. The USGS Indian River stream gage at Rosedale Beach (Gage 01484540) is located less than a mile from the Oak Orchard community, and was used to estimate average low tide, high tide, and overall average water surface elevations for the Indian River Bay (USGS, 2012). The gage data were provided in the National Geodetic Vertical Datum of 1929 (NGVD29) and corrected to NAVD88 by adding -0.78 foot to the NGVD29 elevation. According to daily data from 2006 to 2015, the average low tide elevation is -0.83 foot, the average high tide elevation is 1.83 feet, and the overall average water surface elevation is 0.5 foot. The time between low tide and high tide is approximately 6 hours, while the time between initial hydrograph response and peak discharge is over 8 hours at OO_04. The tidal conditions would therefore be expected to vary from low tide to high tide for an actual event, although a single tide elevation simplification is used for this study.

4.2 Existing Hydraulic Calculations

The culvert hydraulics were calculated using HY-8 Culvert Analysis Program version 7.2 (FHWA, 2012). Rating curves (flow versus stage) were calculated for average low tide, average high tide, and overall average water surface elevations for the Indian River Bay. The rating curve data are provided in Table 2. The HY-8 output is provided in Section 10 with the 25-year design flow and flows from 50 cubic feet per second to 500 cubic feet per second. HY-8 can calculate headwater elevations for only 11 flows at a time, so HY-8 was run with multiple flow ranges to calculate the rating curve provided in Table 2. The HY-8 flow ranges were 0 to 50 cubic feet per second, 50 to 130 cubic feet per second, 130 to 170 cubic feet per second, 170 to 250 cubic feet per second, and 250 to 1000 cubic feet per second.

Table 2: Existing Conditions Rating Curves for Varying Indian River Bay Tailwater Boundary Conditions

Discharge, cubic feet per second	Water Surface Elevation Upstream of River Road for Varying Indian River Bay Tailwater Conditions (feet NAVD88)		
	Tailwater = 0.5 (feet NAVD88)	Tailwater = 1.83 (feet NAVD88)	Tailwater = -0.83 (feet NAVD88)
0	0.5	1.83	-0.83
5	0.96	2.08	0.25
10	2.06	2.15	1.79
15	2.15	2.2	2.14
20	2.2	2.23	2.18
25	2.23	2.26	2.22
30	2.26	2.28	2.25
35	2.28	2.3	2.27
40	2.3	2.32	2.29
45	2.32	2.33	2.31
50	2.33	2.35	2.33
58	2.36	2.37	2.35
66	2.38	2.39	2.37
74	2.4	2.41	2.39
82	2.42	2.43	2.41

Discharge, cubic feet per second	Water Surface Elevation Upstream of River Road for Varying Indian River Bay Tailwater Conditions (feet NAVD88)		
	Tailwater = 0.5 (feet NAVD88)	Tailwater = 1.83 (feet NAVD88)	Tailwater = -0.83 (feet NAVD88)
90	2.43	2.44	2.43
98	2.45	2.46	2.45
106	2.47	2.48	2.47
114	2.48	2.49	2.48
122	2.5	2.51	2.5
130	2.51	2.52	2.51
134	2.52	2.53	2.52
138	2.53	2.54	2.53
142	2.54	2.54	2.53
146	2.54	2.55	2.54
150	2.55	2.56	2.55
154	2.56	2.56	2.56
158	2.56	2.57	2.56
162	2.57	2.58	2.57
166	2.58	2.58	2.58
170	2.58	2.59	2.58
178	2.6	2.6	2.6
186	2.61	2.62	2.61
194	2.62	2.63	2.62
202	2.63	2.64	2.63
210	2.65	2.65	2.64
218	2.66	2.66	2.66
226	2.67	2.68	2.67
234	2.68	2.69	2.68
242	2.69	2.7	2.69
250	2.7	2.71	2.7
325	2.8	2.8	2.8
400	2.89	2.89	2.88
475	2.96	2.97	2.96
550	3.04	3.04	3.04
625	3.11	3.11	3.1
700	3.17	3.17	3.17
775	3.23	3.23	3.23
850	3.28	3.28	3.28
925	3.33	3.34	3.33
1000	3.38	3.38	3.38

The inflow hydrographs from HEC-HMS, the existing stage-storage curves, and the existing conditions rating curves calculated using HY-8, were used to compute outflow hydrographs. The hydrograph routing was performed using the Hydraulic Toolbox version 4.2 (FHWA, 2014). As discussed in Section 3.2, the road begins to be overtopped at elevation 2 feet NAVD88. Table 3 and Table 4 summarize the results of the hydrograph routing. The road is expected to be overtopped for the 2-year storm event. These results are consistent with the questionnaires and conversations with residents in the field that indicated that the road may flood multiple times a year. Figure 5 displays the inflow and outflow hydrographs for the 2-year storm event with the average Indian River Bay water surface elevation (0.5 foot NAVD88). Only the 2-year storm event is displayed in Table 3 because it is the highest frequency storm event that overtops the road.

Table 3 Existing Conditions Peak Flow and Peak Water Surface Elevation Calculated from Hydrograph Routing for 2-year Storm Event with Varying Tailwater Conditions

Tide Description	Tide Elevation (feet NAVD88)	Existing Conditions	
		Peak Flow (cubic feet per second)	Peak Stage (feet NAVD88)
Average Low Tide	-0.83	66.8	2.4[a]
Average Tide	0.5	68.2	2.4[a]
Average High Tide	1.83	69.5	2.4[a]

[a] = Expected to overtop existing roadway

Table 4: Existing Conditions Peak Flow and Peak Water Surface Elevation Calculated from Hydrograph Routing for Average Tide Tailwater Conditions (0.5 foot NAVD88)

Storm Event	Existing Conditions		FEMA Stillwater Elevation[b] (feet NAVD88)
	Peak Flow (cubic feet per second)	Peak Stage (feet NAVD88)	
2-year	70	2.4[a]	Not Provided
10-year	170	2.6[a]	3.8
25-year	250	2.7[a]	Not Provided
50-year	330	2.8[a]	5.6
100-year	410	2.9[a]	6.7

[a] = Expected to overtop existing roadway
[b] = The stillwater elevations for the 10-, 50-, and 100-year storm events are from the Sussex County 2005 Flood Insurance Study (FIS) Report (FEMA, 2005), although the flood maps indicate that the 100-year base flood elevations range from 7 to 9 feet over the study area

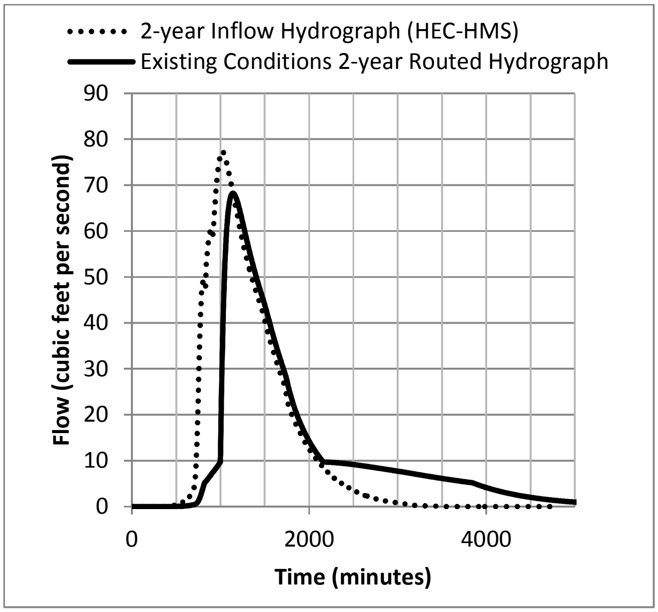

Figure 5: Existing Conditions 2-year Inflow and Routed Hydrograph (0.5 foot NAVD88 tailwater boundary condition)

4.3 Proposed Hydraulic Calculations

The proposed hydraulic conditions were calculated using the same method used for the existing hydraulic conditions. HY-8 was used to compute rating curves for the proposed culvert and bulkhead conditions. An iterative approach was used to calculate the required pipe sizes and bulkhead elevation based on the existing site constraints. The final rating curves (flow versus stage) were calculated for average low tide, average high tide, and overall average water surface elevations for the Indian River Bay. The rating curves are shown in Table 5.

Table 5: Proposed Conditions Rating Curves for Varying Indian River Bay Tailwater Boundary Conditions

Discharge, cubic feet per second	Water Surface Elevation Upstream of Proposed Bulkhead for Varying Indian River Bay Tailwater Conditions (feet NAVD88)		
	Tailwater = 0.5 (feet NAVD88)	Tailwater = 1.83 (feet NAVD88)	Tailwater = -0.83 (feet NAVD88)
0	0.5	1.83	-0.83
5	0.62	1.83	-0.8
10	0.62	1.84	-0.72
15	0.62	1.86	-0.55
20	0.62	1.88	-0.4
25	0.62	1.92	-0.26
30	0.62	1.95	-0.12
35	0.63	2	0.01
40	0.66	2.05	0.14
45	0.71	2.1	0.25
50	0.76	2.17	0.36
58	0.84	2.28	0.53
66	1.09	2.42	0.71
74	1.24	2.57	0.88

Discharge, cubic feet per second	Water Surface Elevation Upstream of Proposed Bulkhead for Varying Indian River Bay Tailwater Conditions (feet NAVD88)		
	Tailwater = 0.5 (feet NAVD88)	Tailwater = 1.83 (feet NAVD88)	Tailwater = -0.83 (feet NAVD88)
82	1.4	2.73	1.06
90	1.59	2.92	1.25
98	1.79	3.12	1.46
106	2.01	3.34	1.68
114	2.25	3.58	1.92
122	2.5	3.83	2.18
130	2.77	4.1	2.46
134	2.91	4.24	2.6
138	3.06	4.39	2.75
142	3.21	4.54	2.91
146	3.37	4.69	3.06
150	3.53	4.85	3.23
154	3.69	5.01	3.39
158	3.86	5.17	3.56
162	4.03	5.3	3.73
166	4.21	5.37	3.91
170	4.39	5.43	4.09
178	4.76	5.51	4.46
186	5.13	5.52	4.84
194	5.37	5.53	5.21
202	5.48	5.54	5.39
210	5.52	5.55	5.5
218	5.53	5.56	5.52
226	5.54	5.56	5.53
234	5.55	5.57	5.54
242	5.55	5.58	5.55
250	5.56	5.58	5.55
325	5.62	5.63	5.61
400	5.66	5.67	5.66
475	5.7	5.71	5.7
550	5.73	5.75	5.73
625	5.77	5.78	5.76
700	5.8	5.81	5.8
775	5.83	5.84	5.82
850	5.86	5.87	5.85
925	5.88	5.89	5.88
1000	5.91	5.92	5.91

The inflow hydrographs from HEC-HMS, the proposed conditions stage-storage curves, and the proposed conditions rating curves calculated using HY-8 were used to compute an outflow hydrograph. The hydrograph routing was performed using the Hydraulic Toolbox version 4.2

(FHWA, 2014). The proposed bulkhead would not be overtopped until water reached an elevation of 5.25 feet NAVD88. Table 6 and Table 7 summarize the results of the hydrograph routing. The proposed bulkhead is expected to be overtopped for the 50-year storm event, but allow the 25-year storm event to be conveyed without impacting River Road, even if it occurred entirely during the average high tide (which is unrealistic given the duration of tides and the anticipated duration of the storm event as discussed in Section 4.1). Figure 6 displays the inflow and proposed outflow hydrographs for the 25-year storm event with the average Indian River Bay water surface elevation (0.5 foot NAVD88).

Table 6: Proposed Conditions Peak Flow and Peak Water Surface Elevation Calculated from Hydrograph Routing for 25-year Storm Event for Varying Tailwater Conditions

Tide Description	Tide Elevation (feet NAVD88)	Proposed Conditions	
		Peak Flow (cubic feet per second)	Peak Stage (feet NAVD88)
Average Low Tide	-0.83	170	4.0
Average Tide	0.5	170	4.2
Average High Tide	1.83	160	5.2

Table 7: Proposed Conditions Peak Flow and Peak Water Surface Elevation Calculated from Hydrograph Routing for Average Tide Tailwater Conditions (0.5 foot NAVD88)

Storm Event	Proposed Conditions		FEMA Stillwater Elevation[b] (feet NAVD88)
	Peak Flow (cubic feet per second)	Peak Stage (feet NAVD88)	
2-year	70	1.1	Not Provided
10-year	130	2.7	3.8
25-year	170	4.2	Not Provided
50-year	220	5.5[a]	5.6
100-year	380	5.6[a]	6.7

[a] = Expected to overtop proposed bulkhead
[b] = The stillwater elevations for the 10-, 50-, and 100-year storm events are from the Sussex County 2005 FIS Report (FEMA, 2005), although the flood maps indicate that the 100-year base flood elevations range from 7 to 9 feet over the study area

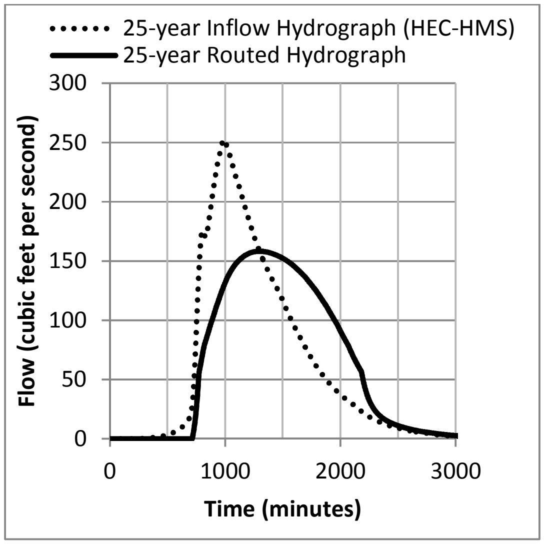

Figure 6: Proposed Conditions 25-year Inflow and Routed Hydrograph (0.5 foot NAVD88 tailwater boundary condition)

5 IMPROVEMENTS AND BENEFITS

The proposed design would reduce the frequency and duration of water ponding on and adjacent to this section of River Road by increasing both conveyance capacity and storage.

When the flow exceeds the existing pipe capacity, the existing storage area includes River Road and nearby properties, and there is no mechanism to drain it. Installing the bulkhead will remove River Road and nearby properties from the storage area for the 10- and 25-year storm events. The three catch basins will allow this area to drain if the tide or the marsh level is below the road elevation. Table 4 and Table 7 provide the existing conditions and proposed water surface elevations along with the stillwater elevations from the FEMA 2005 FIS Report.

The upstream and downstream ends of the existing culvert are in poor condition. Based on the flow observed in the field, it appears the pipe still conveys significant flow, although it will require repair and possibly slip lining. It is recommended that a non-electronic, self-regulating tide gate (e.g., the Waterman self-regulating tide gate) be installed at this location to convey salt water into the marsh for biological habitat during low and average tides, while preventing flow during high tide. A headwall would be required to install a self-regulating tide gate. By installing backwater control check valves for the three proposed 2.5-foot-diameter pipes and a tide gate for the existing 1.5-foot-diameter pipe, the maximum flow rate from the marsh to the Indian River Bay would be substantially higher than the flow rate from the Indian River Bay back into the marsh.

6 FEASIBILITY ASSESSMENT

Soil and Groundwater: The soils at the proposed design site are primarily hydrologic group A, which are well drained and primarily composed of sand (NRCS, 2009). The marsh area is primarily hydrologic group D soils, which are very poorly drained with high clay content. Groundwater data from the Delaware Geologic Survey (2008) suggest that the water table is approximately 1 to 2 feet below the ground surface. When the water table is high, standing water is expected in the marsh area whether or not there is a direct connection to the Indian River Bay.

Construction Access: This site is accessible from River Road, although a portion of the culvert and the bulkhead would be constructed on private property. According to the Sussex County, Delaware GIS parcel database (Sussex County Assessment Office, 2008) approximately 10 easements will be required. This will need to be verified during final design. Construction equipment may need to be parked on the grass area north of River Road, to the west of the existing culvert.

Maintenance Considerations: Routine maintenance of culverts, backwater control check valves, and the tide gate will be required to sustain culvert capacity. Maintenance would include periodically removing sediment and debris from inlets, catch basins, backwater control check valves, and the tide gate. Frequent maintenance of the backwater control check valves and tide gate are critical to ensure successful function of this alternative.

Utility Conflicts: The Sussex County Engineering Department provided as-built plans for the sewer installation at Oak Orchard. No water lines were observed in the vicinity of the project, nor were they indicated on data provided. The proposed culvert would cross a 12-inch PVC sewer line; however, the sewer invert elevations are -3.38 feet NAVD88 and -4.09 feet NAVD88 at the sewer manholes to the south and north of the proposed culvert (not shown on map), respectively. Based on the as-built drawings, the proposed culvert would be installed approximately 1 foot above the sewer line, and therefore no impact to the sewer line is anticipated, although further investigation will be required during detailed design. Above-ground electric lines are located south of River Road at this location and are not anticipated to impact construction. There may be underground cable lines, which will need to be confirmed during detailed design.

Effectiveness: The proposed design would reduce nuisance flooding from frequent storm events on River Road and the nearby properties. Flooding from large coastal events would still be expected; however, the duration of flooding would be reduced. The effectiveness of the proposed design would be dependent on the effective routine maintenance of the proposed culverts, tide gate, and backflow preventer. Additional study would be required to verify that the property of 31803 Betty Jane Lane (see Figure 4) would not be negatively affected by the proposed design, and if it would, additional bulkhead could be installed east of the property. By installing the headwall at a lower elevation than the rest of the bulkhead, an emergency spillway will be created that would convey flow to River Road, where it would naturally disperse (as it would if the bulkhead were not present). This would prevent additional flooding at the structures between the proposed bulkhead and River Road (storage buildings and 31625 River Road).

Environmental Issues: Environmental impacts are expected because construction would occur adjacent or within the marsh. There are approximately 10 to 20 trees along the proposed bulkhead, as well as invasive phragmites. The invasive phragmites can be removed, but a detailed survey will be required to design the bulkhead in a way that limits impact on existing trees. It is possible that wetland vegetation may also be affected, although the removal of the phragmites will benefit native vegetation.

Impact on Base Flood Elevations: The stillwater elevations for the for the 10- 50-, and 100-year-annual-chance recurrence are provided in the FEMA 2005 FIS report for Sussex County (Table 4 and Table 7). These values are all greater than the backwater elevations calculated upstream of the proposed bulkhead. Coastal backwater from the Indian River Bay is therefore still expected to be the predominant control for low-probability events. The proposed design is expected to prevent flooding from higher probability events (2-year to 25-year events) without increasing frequency or duration of flooding from low-probability events (50-year and 100-year events).

7 PLANS AND PERMITTING

Several construction documents and plans would need to be obtained to implement the proposed drainage design, including, but not limited to those described in Table 8.

Table 8: Required Plans and Permitting for Proposed OO_04 Design

Plans/Permits	Permitting Agency	Notes and Potential Difficulties
Wetlands and Subaqueous Lands Permit	DNREC	The proposed culvert and bulkhead will impact the marsh north of the River Road.
Traffic Control Plan	DelDOT	River Road will be impacted while the proposed culverts and catch basins are installed.
Erosion and Sediment Control Plan	County Conservation District	
Utility Construction Permit	DelDOT	Limited utility impacts are anticipated for this project, although care will need to be taken to avoid damaging the existing sewer line.

8 COST ESTIMATE

Table 9 summarizes the costs associated with this concept design.

Table 9: Estimated Project Costs for OO_04

ITEM	QUANTITY	UNITS	UNIT COST	TOTAL
Excavation	1100	CY	$25.00	$27,500
Grading	220	SY	$2.50	$550
Bulkhead (Reinforced Concrete)	500	CY	$800.00	$400,000
Asphalt Base	6	TON	$100.00	$600
Asphalt Surface	16	TON	$110.00	$1,760
Hydroseeding	80	SY	$0.75	$60
15" Tideflex CheckMate Valve	3	EA	$3,300.00	$9,900
30" Backflow Control Check Valve	3	EA	$7,040.00	$21,120
18" Self-Regulating Tide Gate	1	EA	$30,000.00	$30,000
Inlet	3	EA	$2,500.00	$7,500
Junction Box	1	EA	$2,500.00	$2,500
Traffic Control Plan	7	DAY	$750.00	$5,250
Remove and Dispose Existing Headwall	5	SY	$12.00	$60
Slip Line Pipe	140	LF	$59.00	$8,260
Endwall (18" pipe)	1	EA	$1,400.00	$1,400
Endwall (30" pipe)	6	EA	$2,500.00	$15,000
15" High Density Polyethylene Pipe	140	LF	$45.00	$6,300
30" High Density Polyethylene Pipe	410	LF	$70.00	$28,700

Initial Project Costs		$569,610
Contingency	10%	$56,961
Erosion and Sediment Control	10%	$56,961
Base Construction Costs		$683,532
Mobilization	5%	$34,177
Subtotal 1		$717,709
Contingency	15%	$107,656
Subtotal 2		$825,365
Engineering		$120,000
Total		$945,365

CY = cubic yard
EA = each
LF = linear foot
SY = square yard
TON = ton

9 REFERENCES

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10 HY-8 REPORT

Table HY8- 1 - Summary of Culvert Flows at Crossing: OO_04B_Proposed_Average_Tide

Headwater Elevation (ft)	Total Discharge (cfs)	OO_04 Proposed (Proposed Average Tide Model) Discharge (cfs)	OO_04_Existing Average Tide (Proposed Average Tide Model) Discharge (cfs)	Roadway Discharge (cfs)	Iterations
0.76	50.00	46.40	3.74	0.00	11
1.71	95.00	86.88	8.16	0.00	3
3.14	140.00	128.00	12.03	0.00	4
4.20	166.00	151.74	14.26	0.00	4
5.54	230.00	177.00	16.64	35.43	17
5.58	275.00	177.70	16.70	79.77	5
5.61	320.00	178.26	16.76	124.08	4
5.64	365.00	178.73	16.80	167.41	3
5.67	410.00	179.17	16.84	212.55	3
5.69	455.00	179.58	16.88	257.70	3
5.71	500.00	179.97	16.92	302.67	3
5.25	187.95	171.80	16.15	0.00	Overtopping

Table HY8- 2 - Culvert Summary Table: Proposed_Average_Tide

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	46.40	0.76	1.912	2.255	1-S1f	1.016	1.322	2.500	3.500	3.151	0.000
95.00	86.88	1.71	2.903	3.215	4-FFf	1.469	1.828	2.500	3.500	5.900	0.000
140.00	128.00	3.14	4.346	4.636	4-FFf	1.966	2.169	2.500	3.500	8.692	0.000
166.00	151.74	4.20	5.422	5.705	4-FFf	2.500	2.336	2.500	3.500	10.304	0.000
230.00	177.00	5.54	6.719	7.041	4-FFf	2.500	2.500	2.500	3.500	12.020	0.000
275.00	177.70	5.58	6.757	7.081	4-FFf	2.500	2.500	2.500	3.500	12.067	0.000
320.00	178.26	5.61	6.787	7.113	4-FFf	2.500	2.500	2.500	3.500	12.105	0.000
365.00	178.73	5.64	6.813	7.140	4-FFf	2.500	2.500	2.500	3.500	12.137	0.000
410.00	179.17	5.67	6.837	7.165	4-FFf	2.500	2.500	2.500	3.500	12.167	0.000
455.00	179.58	5.69	6.859	7.189	4-FFf	2.500	2.500	2.500	3.500	12.195	0.000
500.00	179.97	5.71	6.881	7.211	4-FFf	2.500	2.500	2.500	3.500	12.221	0.000

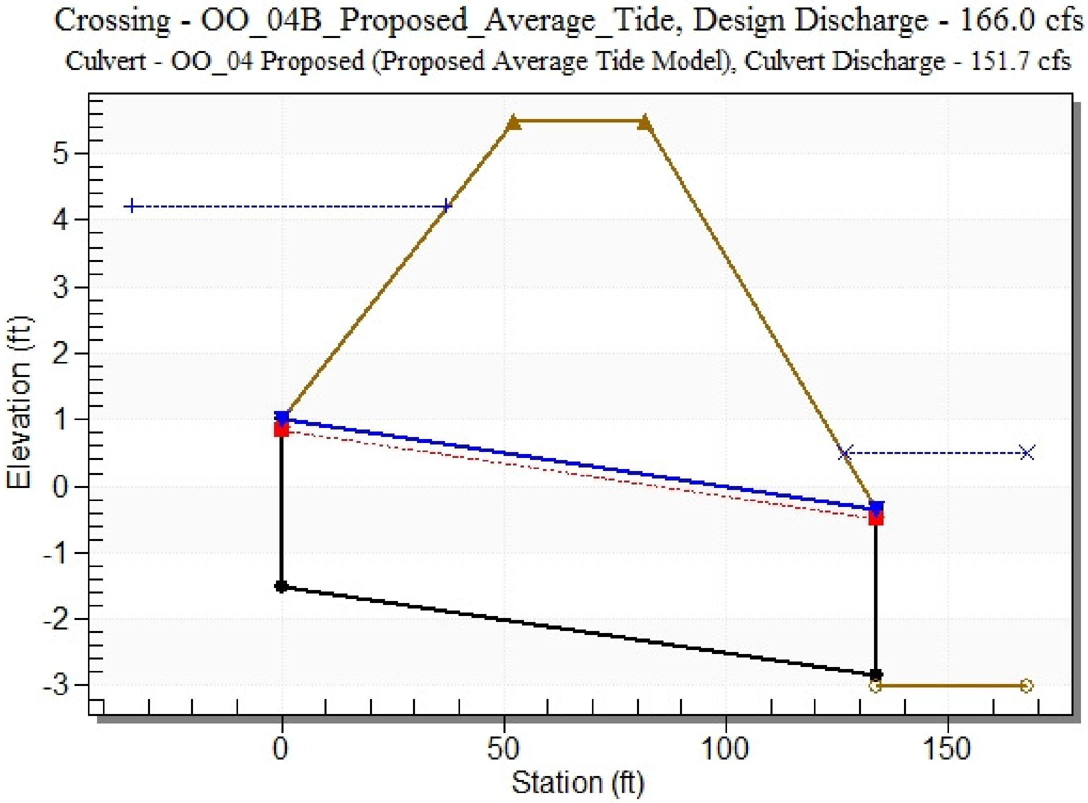
****

Inlet Elevation (invert): -1.50 ft,   Outlet Elevation (invert): -2.84 ft

Culvert Length: 134.01 ft,   Culvert Slope: 0.0100

****

Water Surface Profile Plot for Culvert: OO_04 Proposed (Proposed Average Tide Model)



Site Data - OO_04 Proposed (Proposed Average Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.50 ft

Outlet Station: 134.00 ft

Outlet Elevation: -2.84 ft

Number of Barrels: 3

Culvert Data Summary - OO_04 Proposed (Proposed Average Tide Model)

Barrel Shape: Circular

Barrel Diameter: 2.50 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Beveled Edge (1:1)

Inlet Depression: NONE

Table HY8- 3 - Culvert Summary Table: OO_04_Existing_Average_Tide (Proposed Average Tide Model)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	3.74	0.76	1.087	1.955	4-FFf	1.082	0.738	1.500	3.500	2.116	0.000
95.00	8.16	1.71	1.858	2.914	4-FFf	1.500	1.103	1.500	3.500	4.620	0.000
140.00	12.03	3.14	2.832	4.335	4-FFf	1.500	1.307	1.500	3.500	6.808	0.000
166.00	14.26	4.20	3.584	5.404	4-FFf	1.500	1.409	1.500	3.500	8.071	0.000
230.00	16.64	5.54	4.527	6.741	4-FFf	1.500	1.500	1.500	3.500	9.415	0.000
275.00	16.70	5.58	4.557	6.781	4-FFf	1.500	1.500	1.500	3.500	9.453	0.000
320.00	16.76	5.61	4.581	6.812	4-FFf	1.500	1.500	1.500	3.500	9.482	0.000
365.00	16.80	5.64	4.601	6.839	4-FFf	1.500	1.500	1.500	3.500	9.507	0.000
410.00	16.84	5.67	4.620	6.865	4-FFf	1.500	1.500	1.500	3.500	9.531	0.000
455.00	16.88	5.69	4.638	6.889	4-FFf	1.500	1.500	1.500	3.500	9.553	0.000
500.00	16.92	5.71	4.655	6.911	4-FFf	1.500	1.500	1.500	3.500	9.573	0.000

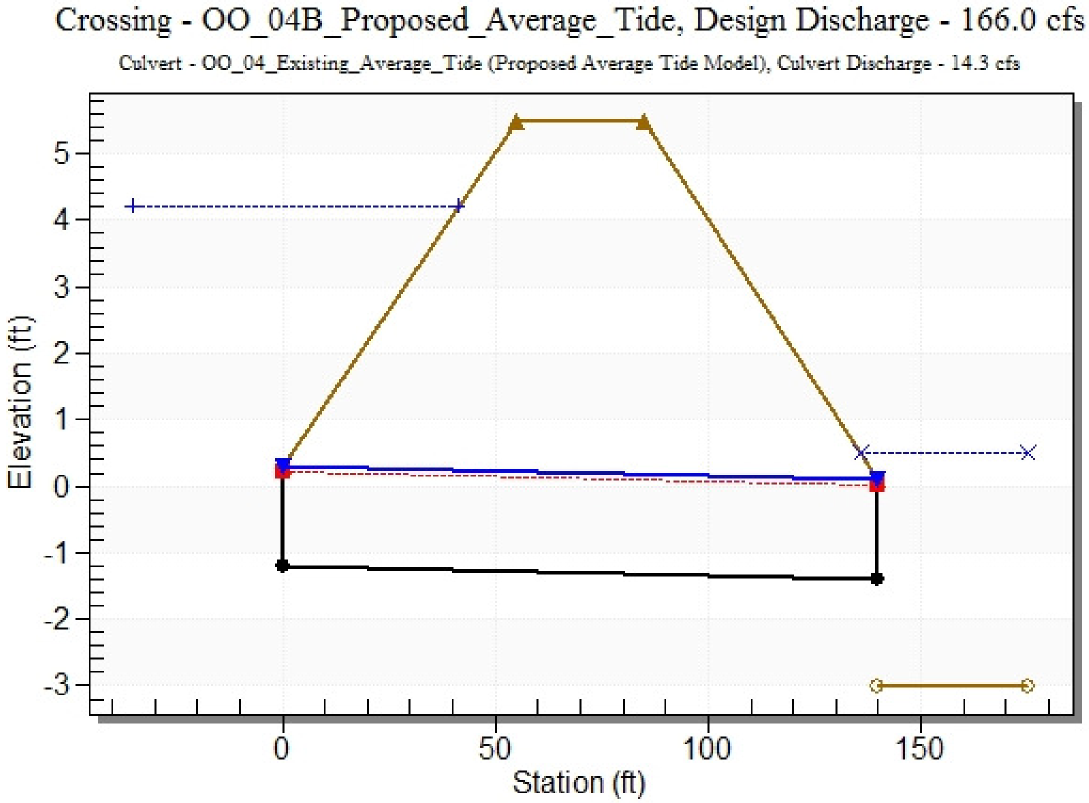
****

Inlet Elevation (invert): -1.20 ft, Outlet Elevation (invert): -1.40 ft

Culvert Length: 140.00 ft, Culvert Slope: 0.0014

****

Water Surface Profile Plot for Culvert: OO_04 Existing (Proposed Average Tide Model)



Site Data - OO_04_Existing_Average_Tide (Proposed Average Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.20 ft

Outlet Station: 140.00 ft

Outlet Elevation: -1.40 ft

Number of Barrels: 1

Culvert Data Summary - OO_04_Existing_Average_Tide (Proposed Average Tide Model)

Barrel Shape: Circular

Barrel Diameter: 1.50 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: NONE

Table HY8- 4 - Downstream Channel Rating Curve (Crossing: OO_04B_Proposed_Average_Tide)OO_04B_Proposed_Average_Tide)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
50.00	0.50	3.50
95.00	0.50	3.50
140.00	0.50	3.50
166.00	0.50	3.50
230.00	0.50	3.50
275.00	0.50	3.50
320.00	0.50	3.50
365.00	0.50	3.50
410.00	0.50	3.50
455.00	0.50	3.50
500.00	0.50	3.50

Tailwater Channel Data - OO_04B_Proposed_Average_Tide

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 0.50 ft

Roadway Data for Crossing: OO_04B_Proposed_Average_Tide

Roadway Profile Shape: Irregular Roadway Shape (coordinates)

Irregular Roadway Cross-Section:

Coord No.	Station (ft)	Elevation (ft)
0	0.00	5.50
1	650.70	5.50
2	650.70	5.25
3	675.70	5.25
4	675.70	5.50
5	990.00	5.50

Roadway Surface: Paved

Roadway Top Width: 30.00 ft

Table HY8- 5 - Summary of Culvert Flows at Crossing: OO_04B_Proposed_Low_Tide

Headwater Elevation (ft)	Total Discharge (cfs)	OO_04 Proposed (Proposed Low Tide Model) Discharge (cfs)	OO_04_Existing_Low_Tide (Proposed Low Tide Model) Discharge (cfs)	Roadway Discharge (cfs)	Iterations
0.36	50.00	44.44	5.56	0.00	4
1.38	95.00	86.08	8.92	0.00	4
2.83	140.00	127.63	12.39	0.00	4
3.96	167.00	152.42	14.59	0.00	4
5.53	230.00	182.74	17.27	28.47	23
5.58	275.00	183.51	17.34	73.08	5
5.61	320.00	184.10	17.39	117.46	4
5.64	365.00	184.61	17.44	162.50	4
5.66	410.00	185.06	17.48	206.17	3
5.69	455.00	185.49	17.51	251.12	3
5.71	500.00	185.89	17.55	296.07	3
5.25	194.37	177.55	16.82	0.00	Overtopping

Table HY8- 6 - Culvert Summary Table: OO_04 Proposed (Proposed Low Tide Model)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	44.44	0.36	1.865	0.0*	1-S2n	0.992	1.294	0.993	2.170	8.153	0.000
95.00	86.08	1.38	2.880	0.0*	5-S2n	1.460	1.820	1.464	2.170	9.611	0.000
140.00	127.63	2.83	4.331	0.0*	5-S2n	1.961	2.166	1.965	2.170	10.289	0.000
167.00	152.42	3.96	5.455	4.846	7-M2c	2.500	2.341	2.314	2.170	10.767	0.000
230.00	182.74	5.53	7.033	6.534	6-FFc	2.500	2.500	2.500	2.170	12.409	0.000
275.00	183.51	5.58	7.076	6.579	6-FFc	2.500	2.500	2.500	2.170	12.462	0.000
320.00	184.10	5.61	7.108	6.614	6-FFc	2.500	2.500	2.500	2.170	12.501	0.000
365.00	184.61	5.64	7.137	6.644	6-FFc	2.500	2.500	2.500	2.170	12.536	0.000
410.00	185.06	5.66	7.162	6.671	6-FFc	2.500	2.500	2.500	2.170	12.567	0.000
455.00	185.49	5.69	7.186	6.696	6-FFc	2.500	2.500	2.500	2.170	12.596	0.000
500.00	185.89	5.71	7.208	6.720	6-FFc	2.500	2.500	2.500	2.170	12.623	0.000

* theoretical depth is impractical. Depth reported is corrected.

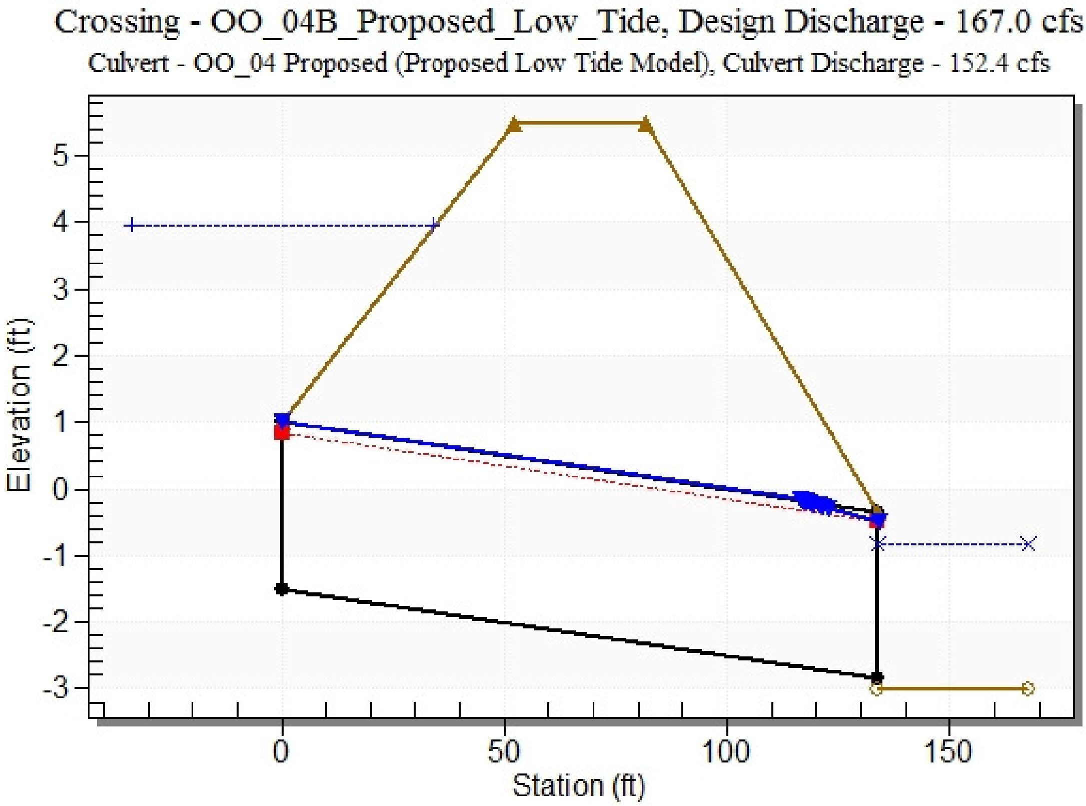
****

Inlet Elevation (invert): -1.50 ft, Outlet Elevation (invert): -2.84 ft

Culvert Length: 134.01 ft, Culvert Slope: 0.0100

****

Water Surface Profile Plot for Culvert: OO_04 Proposed (Proposed Low Tide Model)



Site Data - OO_04 Proposed (Proposed Low Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.50 ft

Outlet Station: 134.00 ft

Outlet Elevation: -2.84 ft

Number of Barrels: 3

Culvert Data Summary - OO_04 Proposed (Proposed Low Tide Model)

Barrel Shape: Circular

Barrel Diameter: 2.50 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Beveled Edge (1:1)

Inlet Depression: NONE

Table HY8- 7 - Culvert Summary Table: OO_04_Existing_Low_Tide (Proposed Low Tide Model)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	5.56	0.36	1.392	1.564	2-M2c	1.500	0.909	0.909	2.170	4.963	0.000
95.00	8.92	1.38	2.018	2.579	7-M2c	1.500	1.152	1.156	2.170	6.106	0.000
140.00	12.39	2.83	2.943	4.030	7-M2c	1.500	1.324	1.332	2.170	7.471	0.000
167.00	14.59	3.96	3.704	5.155	7-M2c	1.500	1.423	1.427	2.170	8.406	0.000
230.00	17.27	5.53	4.821	6.733	6-FFc	1.500	1.500	1.500	2.170	9.775	0.000
275.00	17.34	5.58	4.853	6.775	6-FFc	1.500	1.500	1.500	2.170	9.813	0.000
320.00	17.39	5.61	4.877	6.808	6-FFc	1.500	1.500	1.500	2.170	9.842	0.000
365.00	17.44	5.64	4.898	6.836	6-FFc	1.500	1.500	1.500	2.170	9.867	0.000
410.00	17.48	5.66	4.917	6.861	6-FFc	1.500	1.500	1.500	2.170	9.890	0.000
455.00	17.51	5.69	4.935	6.885	6-FFc	1.500	1.500	1.500	2.170	9.911	0.000
500.00	17.55	5.71	4.952	6.908	6-FFc	1.500	1.500	1.500	2.170	9.931	0.000

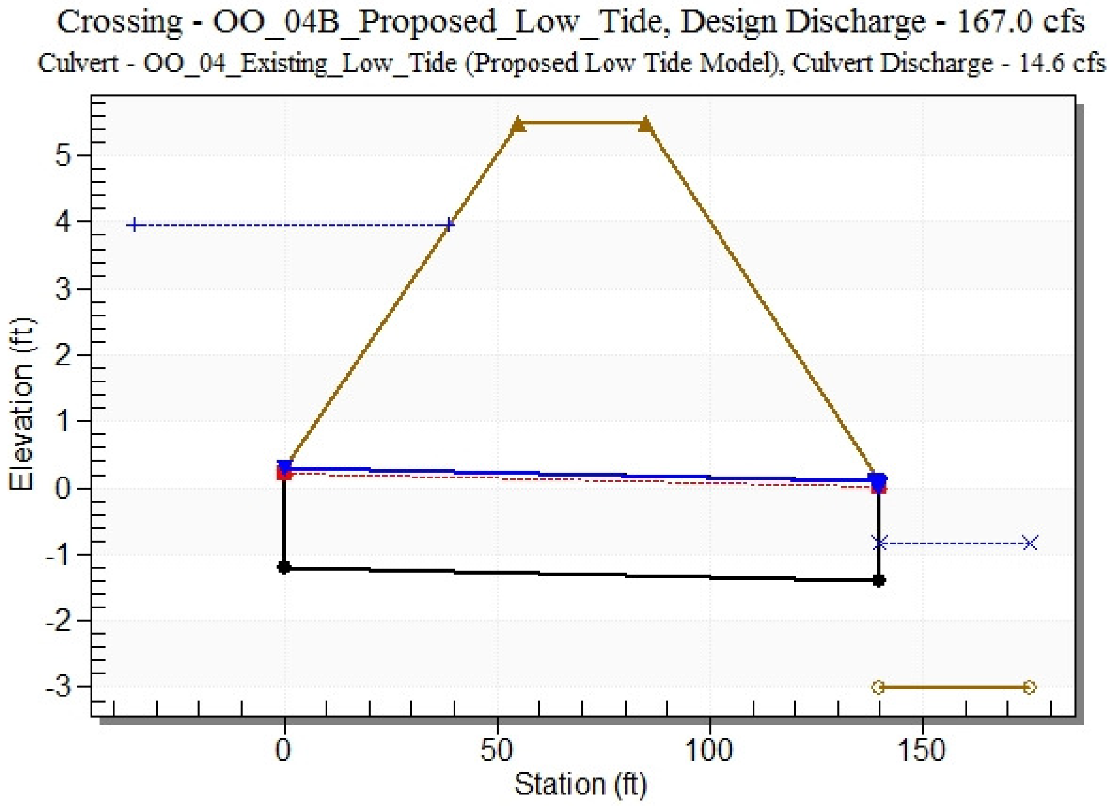
****

Inlet Elevation (invert): -1.20 ft, Outlet Elevation (invert): -1.40 ft

Culvert Length: 140.00 ft, Culvert Slope: 0.0014

****

Water Surface Profile Plot for Culvert: OO_04 Existing (Proposed Low Tide Model)



Site Data - OO_04_Existing_Low_Tide (Proposed Low Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.20 ft

Outlet Station: 140.00 ft

Outlet Elevation: -1.40 ft

Number of Barrels: 1

Culvert Data Summary - OO_04_Existing_Low_Tide (Proposed Low Tide Model)

Barrel Shape: Circular

Barrel Diameter: 1.50 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: NONE

Table HY8- 8 - Downstream Channel Rating Curve (Crossing: OO_04B_Proposed_Low_Tide)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
50.00	-0.83	2.17
95.00	-0.83	2.17
140.00	-0.83	2.17
167.00	-0.83	2.17
230.00	-0.83	2.17
275.00	-0.83	2.17
320.00	-0.83	2.17
365.00	-0.83	2.17
410.00	-0.83	2.17
455.00	-0.83	2.17
500.00	-0.83	2.17

Tailwater Channel Data - OO_04B_Proposed_Low_Tide

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: -0.83 ft

Roadway Data for Crossing: OO_04B_Proposed_Low_Tide

Roadway Profile Shape: Irregular Roadway Shape (coordinates)

Irregular Roadway Cross-Section:

Coord No.	Station (ft)	Elevation (ft)
0	0.00	5.50
1	650.70	5.50
2	650.70	5.25
3	675.70	5.25
4	675.70	5.50
5	990.00	5.50

Roadway Surface: Paved

Roadway Top Width: 30.00 ft

Table HY8- 9 - Summary of Culvert Flows at Crossing: OO_04B_Proposed_High_Tide

Headwater Elevation (ft)	Total Discharge (cfs)	OO_04 Proposed (Proposed High Tide Model) Discharge (cfs)	OO_04_Existing_High_Tide (Proposed High Tide Model) Discharge (cfs)	Roadway Discharge (cfs)	Iterations
2.17	50.00	45.81	4.30	0.00	10
3.04	95.00	86.85	8.16	0.00	5
4.47	140.00	128.00	12.03	0.00	4
5.23	159.00	145.30	13.66	0.00	9
5.57	230.00	152.40	14.33	62.50	11
5.60	275.00	153.08	14.39	106.25	4
5.63	320.00	153.68	14.45	151.31	4
5.66	365.00	154.19	14.49	194.83	3
5.68	410.00	154.68	14.54	239.76	3
5.70	455.00	155.14	14.58	284.70	3
5.72	500.00	155.57	14.62	329.51	3
5.25	159.48	145.78	13.70	0.00	Overtopping

Table HY8- 10 - Culvert Summary Table: OO_04 Proposed (Proposed High Tide Model)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	45.81	2.17	1.898	3.668	4-FFf	1.009	1.314	2.500	4.830	3.110	0.000
95.00	86.85	3.04	2.902	4.544	4-FFf	1.469	1.828	2.500	4.830	5.898	0.000
140.00	128.00	4.47	4.346	5.966	4-FFf	1.966	2.169	2.500	4.830	8.692	0.000
159.00	145.30	5.23	5.115	6.727	4-FFf	2.500	2.291	2.500	4.830	9.867	0.000
230.00	152.40	5.57	5.454	7.067	4-FFf	2.500	2.341	2.500	4.830	10.349	0.000
275.00	153.08	5.60	5.488	7.101	4-FFf	2.500	2.345	2.500	4.830	10.395	0.000
320.00	153.68	5.63	5.517	7.130	4-FFf	2.500	2.350	2.500	4.830	10.436	0.000
365.00	154.19	5.66	5.542	7.156	4-FFf	2.500	2.353	2.500	4.830	10.471	0.000
410.00	154.68	5.68	5.566	7.180	4-FFf	2.500	2.357	2.500	4.830	10.504	0.000
455.00	155.14	5.70	5.589	7.203	4-FFf	2.500	2.360	2.500	4.830	10.535	0.000
500.00	155.57	5.72	5.610	7.224	4-FFf	2.500	2.363	2.500	4.830	10.564	0.000

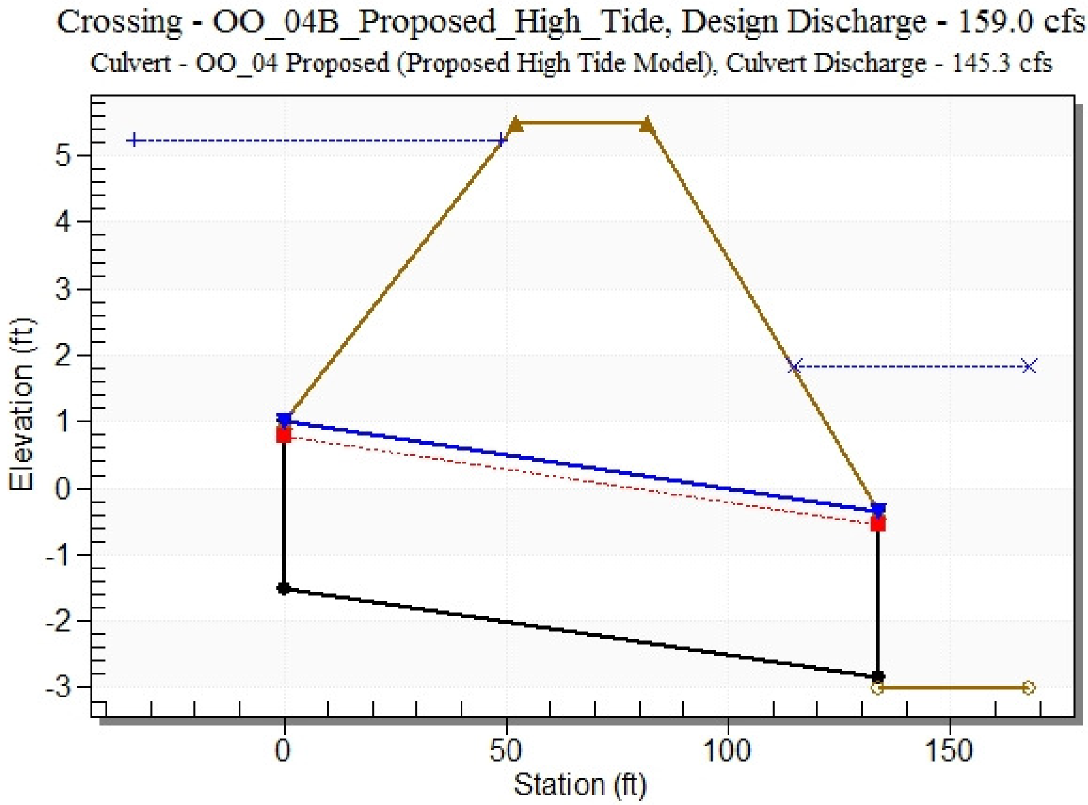
****

Inlet Elevation (invert): -1.50 ft, Outlet Elevation (invert): -2.84 ft

Culvert Length: 134.01 ft, Culvert Slope: 0.0100

****

Water Surface Profile Plot for Culvert: OO_04 Proposed (Proposed High Tide Model)



Site Data - OO_04 Proposed (Proposed High Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.50 ft

Outlet Station: 134.00 ft

Outlet Elevation: -2.84 ft

Number of Barrels: 3

Culvert Data Summary - OO_04 Proposed (Proposed High Tide Model)

Barrel Shape: Circular

Barrel Diameter: 2.50 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Beveled Edge (1:1)

Inlet Depression: NONE

Table HY8- 11 - Culvert Summary Table: OO_04_Existing_High_Tide (Proposed High Tide Model)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	4.30	2.17	1.184	3.367	4-FFf	1.234	0.792	1.500	4.830	2.433	0.000
95.00	8.16	3.04	1.858	4.243	4-FFf	1.500	1.103	1.500	4.830	4.619	0.000
140.00	12.03	4.47	2.832	5.665	4-FFf	1.500	1.307	1.500	4.830	6.808	0.000
159.00	13.66	5.23	3.366	6.426	4-FFf	1.500	1.381	1.500	4.830	7.729	0.000
230.00	14.33	5.57	3.607	6.767	4-FFf	1.500	1.412	1.500	4.830	8.106	0.000
275.00	14.39	5.60	3.631	6.800	4-FFf	1.500	1.414	1.500	4.830	8.143	0.000
320.00	14.45	5.63	3.651	6.830	4-FFf	1.500	1.417	1.500	4.830	8.174	0.000
365.00	14.49	5.66	3.669	6.855	4-FFf	1.500	1.419	1.500	4.830	8.202	0.000
410.00	14.54	5.68	3.687	6.880	4-FFf	1.500	1.421	1.500	4.830	8.228	0.000
455.00	14.58	5.70	3.703	6.902	4-FFf	1.500	1.423	1.500	4.830	8.252	0.000
500.00	14.62	5.72	3.718	6.924	4-FFf	1.500	1.425	1.500	4.830	8.275	0.000

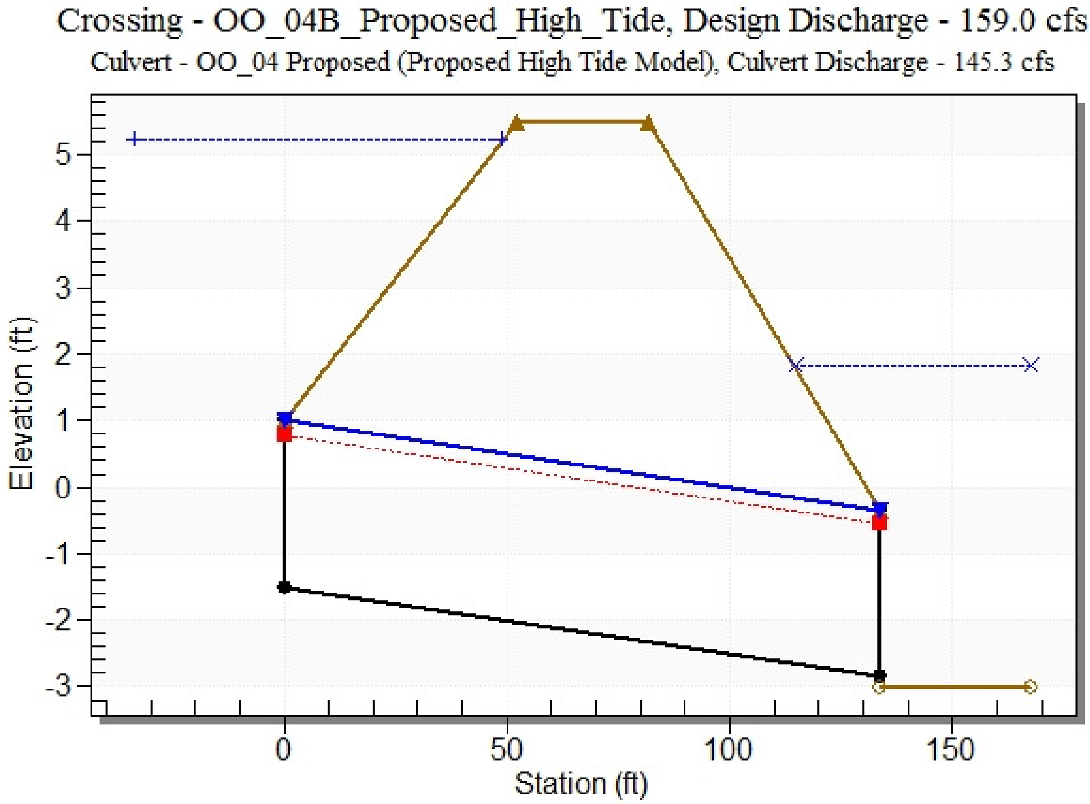
****

Inlet Elevation (invert): -1.20 ft, Outlet Elevation (invert): -1.40 ft

Culvert Length: 140.00 ft, Culvert Slope: 0.0014

****

Water Surface Profile Plot for Culvert: OO_04 Proposed (Proposed High Tide Model)



Site Data - OO_04_Existing_High_Tide (Proposed High Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.20 ft

Outlet Station: 140.00 ft

Outlet Elevation: -1.40 ft

Number of Barrels: 1

Culvert Data Summary - OO_04_Existing_High_Tide (Proposed High Tide Model)

Barrel Shape: Circular

Barrel Diameter: 1.50 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: NONE

Table HY8- 12 - Downstream Channel Rating Curve (Crossing: OO_04B_Proposed_High_Tide)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
50.00	1.83	4.83
95.00	1.83	4.83
140.00	1.83	4.83
158.00	1.83	4.83
230.00	1.83	4.83
275.00	1.83	4.83
320.00	1.83	4.83
365.00	1.83	4.83
410.00	1.83	4.83
455.00	1.83	4.83
500.00	1.83	4.83

Tailwater Channel Data - OO_04B_Proposed_High_Tide

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 1.83 ft

Roadway Data for Crossing: OO_04B_Proposed_High_Tide

Roadway Profile Shape: Irregular Roadway Shape (coordinates)

Irregular Roadway Cross-Section:

Coord No.	Station (ft)	Elevation (ft)
0	0.00	5.50
1	650.70	5.50
2	650.70	5.25
3	675.70	5.25
4	675.70	5.50
5	990.00	5.50

Roadway Surface: Paved

Roadway Top Width: 30.00 ft

Table HY8- 13 - Summary of Culvert Flows at Crossing: OO_4B_Existing_Average_Tide

Headwater Elevation (ft)	Total Discharge (cfs)	OO_04 Existing (Existing Average Tide Model) Discharge (cfs)	Roadway Discharge (cfs)	Iterations
2.33	50.00	10.03	39.84	9
2.44	95.00	10.33	84.19	6
2.53	140.00	10.56	129.00	5
2.61	185.00	10.76	173.57	4
2.68	230.00	10.93	218.70	4
2.70	250.00	11.00	238.33	3
2.79	320.00	11.22	308.66	4
2.85	365.00	11.35	353.32	3
2.90	410.00	11.47	398.38	3
2.94	455.00	11.59	443.37	3
2.99	500.00	11.69	488.05	2
2.00	9.07	9.07	0.00	Overtopping

Table HY8- 14 - Culvert Summary Table: OO_04 Existing (Existing Average Tide Model)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	10.03	2.33	2.277	3.532	4-FFf	1.500	1.217	1.500	2.500	5.677	0.000
95.00	10.33	2.44	2.353	3.643	4-FFf	1.500	1.230	1.500	2.500	5.846	0.000
140.00	10.56	2.53	2.413	3.731	4-FFf	1.500	1.241	1.500	2.500	5.977	0.000
185.00	10.76	2.61	2.465	3.807	4-FFf	1.500	1.250	1.500	2.500	6.087	0.000
230.00	10.93	2.68	2.512	3.875	4-FFf	1.500	1.257	1.500	2.500	6.184	0.000
250.00	11.00	2.70	2.531	3.902	4-FFf	1.500	1.261	1.500	2.500	6.223	0.000
320.00	11.22	2.79	2.594	3.993	4-FFf	1.500	1.271	1.500	2.500	6.350	0.000
365.00	11.35	2.85	2.630	4.046	4-FFf	1.500	1.277	1.500	2.500	6.423	0.000
410.00	11.47	2.90	2.665	4.096	4-FFf	1.500	1.282	1.500	2.500	6.492	0.000
455.00	11.59	2.94	2.698	4.144	4-FFf	1.500	1.287	1.500	2.500	6.556	0.000
500.00	11.69	2.99	2.730	4.189	4-FFf	1.500	1.292	1.500	2.500	6.616	0.000

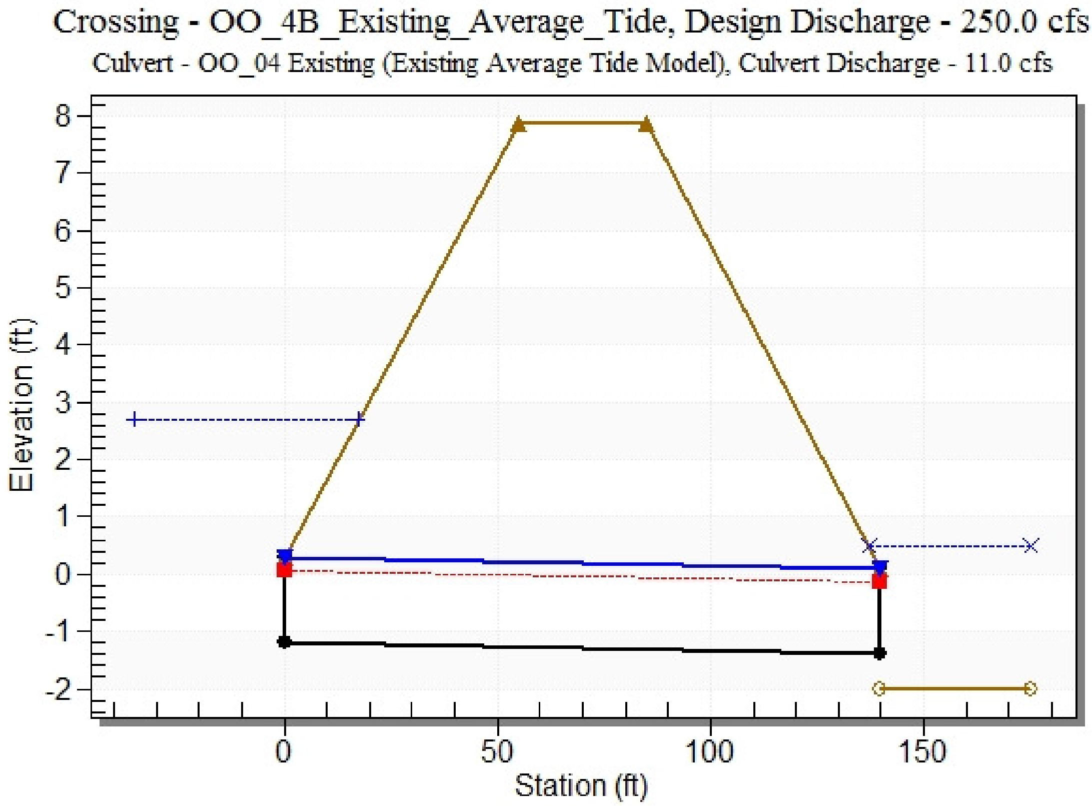
****

Inlet Elevation (invert): -1.20 ft, Outlet Elevation (invert): -1.40 ft

Culvert Length: 140.00 ft, Culvert Slope: 0.0014

****

Water Surface Profile Plot for Culvert: OO_04 Existing (Existing Average Tide Model)



Site Data - OO_04 Existing (Existing Average Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.20 ft

Outlet Station: 140.00 ft

Outlet Elevation: -1.40 ft

Number of Barrels: 1

Culvert Data Summary - OO_04 Existing (Existing Average Tide Model)

Barrel Shape: Circular

Barrel Diameter: 1.50 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: NONE

Table HY8- 15 - Downstream Channel Rating Curve (Crossing: OO_4B_Existing_Average_Tide)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
50.00	0.50	2.50
95.00	0.50	2.50
140.00	0.50	2.50
185.00	0.50	2.50
230.00	0.50	2.50
250.00	0.50	2.50
320.00	0.50	2.50
365.00	0.50	2.50
410.00	0.50	2.50
455.00	0.50	2.50
500.00	0.50	2.50

Tailwater Channel Data - OO_4B_Existing_Average_Tide

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 0.50 ft

Roadway Data for Crossing: OO_4B_Existing_Average_Tide

Roadway Profile Shape: Irregular Roadway Shape (coordinates)

Irregular Roadway Cross-Section:

Coord No.	Station (ft)	Elevation (ft)
0	0.00	7.88
1	98.00	5.89
2	157.00	3.84
3	229.00	3.20
4	393.00	2.00
5	537.00	2.32
6	601.00	3.88
7	636.00	5.53
8	663.00	5.97

Roadway Surface: Paved

Roadway Top Width: 30.00 ft

Table HY8- 16 - Summary of Culvert Flows at Crossing: OO_4B_Existing_Low_Tide

Headwater Elevation (ft)	Total Discharge (cfs)	OO_04 Existing (Existing Low Tide Model) Discharge (cfs)	Roadway Discharge (cfs)	Iterations
2.33	50.00	11.28	38.40	13
2.44	95.00	11.54	82.95	6
2.53	140.00	11.74	127.81	5
2.61	185.00	11.90	172.41	4
2.67	230.00	12.05	217.58	4
2.70	250.00	12.11	237.21	3
2.79	320.00	12.31	307.57	4
2.85	365.00	12.42	352.25	3
2.90	410.00	12.53	397.32	3
2.94	455.00	12.63	442.32	3
2.99	500.00	12.73	486.99	2
2.00	10.53	10.53	0.00	Overtopping

Table HY8- 17 - Culvert Summary Table: OO_04 Existing (Existing Low Tide Model)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	11.28	2.33	2.610	3.528	7-M2c	1.500	1.273	1.282	1.170	7.012	0.000
95.00	11.54	2.44	2.684	3.641	7-M2c	1.500	1.285	1.293	1.170	7.121	0.000
140.00	11.74	2.53	2.743	3.729	7-M2c	1.500	1.294	1.301	1.170	7.209	0.000
185.00	11.90	2.61	2.791	3.805	7-M2c	1.500	1.301	1.311	1.170	7.264	0.000
230.00	12.05	2.67	2.837	3.873	7-M2c	1.500	1.308	1.317	1.170	7.327	0.000
250.00	12.11	2.70	2.856	3.901	7-M2c	1.500	1.311	1.320	1.170	7.352	0.000
320.00	12.31	2.79	2.917	3.992	7-M2c	1.500	1.320	1.328	1.170	7.436	0.000
365.00	12.42	2.85	2.953	4.045	7-M2c	1.500	1.325	1.333	1.170	7.485	0.000
410.00	12.53	2.90	2.988	4.095	7-M2c	1.500	1.330	1.333	1.170	7.550	0.000
455.00	12.63	2.94	3.020	4.143	7-M2c	1.500	1.335	1.336	1.170	7.596	0.000
500.00	12.73	2.99	3.051	4.188	7-M2c	1.500	1.339	1.340	1.170	7.641	0.000

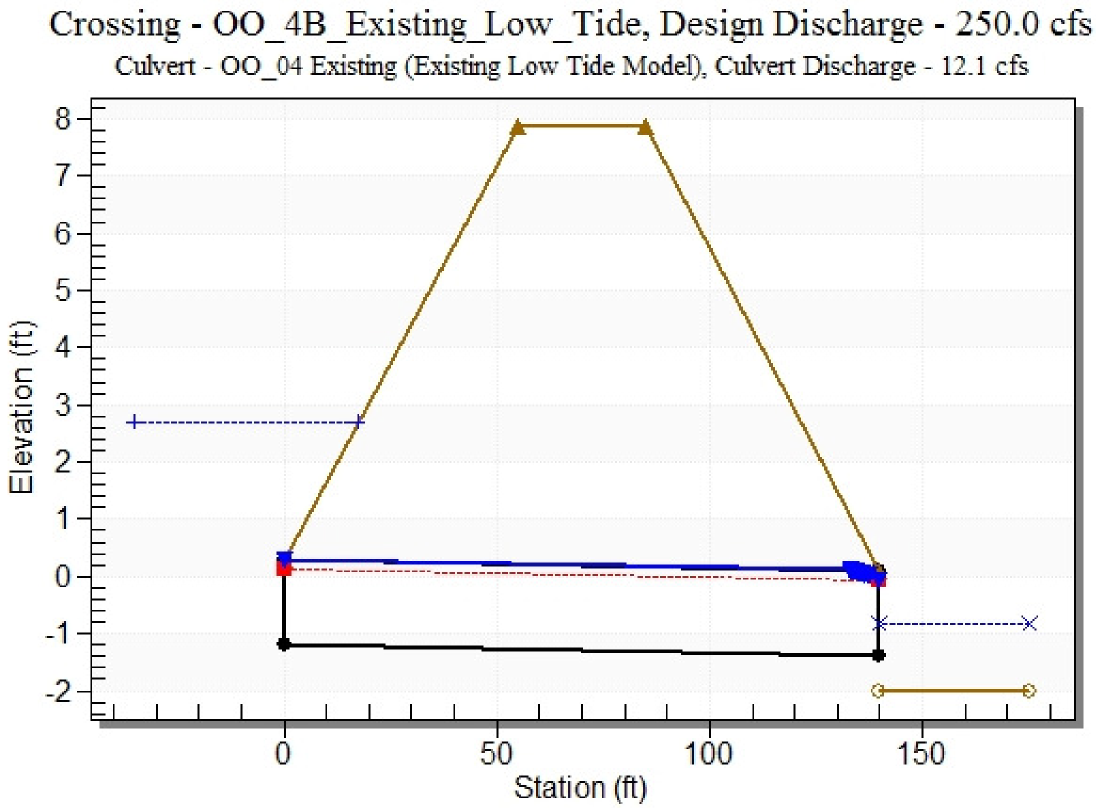
****

Inlet Elevation (invert): -1.20 ft,   Outlet Elevation (invert): -1.40 ft

Culvert Length: 140.00 ft,   Culvert Slope: 0.0014

****

Water Surface Profile Plot for Culvert: OO_04 Existing (Existing Low Tide Model)



Site Data - OO_04 Existing (Existing Low Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.20 ft

Outlet Station: 140.00 ft

Outlet Elevation: -1.40 ft

Number of Barrels: 1

Culvert Data Summary - OO_04 Existing (Existing Low Tide Model)

Barrel Shape: Circular

Barrel Diameter: 1.50 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: NONE

Table HY8- 18 - Downstream Channel Rating Curve (Crossing: OO_4B_Existing_Low_Tide)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
50.00	-0.83	1.17
95.00	-0.83	1.17
140.00	-0.83	1.17
185.00	-0.83	1.17
230.00	-0.83	1.17
250.00	-0.83	1.17
320.00	-0.83	1.17
365.00	-0.83	1.17
410.00	-0.83	1.17
455.00	-0.83	1.17
500.00	-0.83	1.17

Tailwater Channel Data - OO_4B_Existing_Low_Tide

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: -0.83 ft

Roadway Data for Crossing: OO_4B_Existing_Low_Tide

Roadway Profile Shape: Irregular Roadway Shape (coordinates)

Irregular Roadway Cross-Section:

Coord No.	Station (ft)	Elevation (ft)
0	0.00	7.88
1	98.00	5.89
2	157.00	3.84
3	229.00	3.20
4	393.00	2.00
5	537.00	2.32
6	601.00	3.88
7	636.00	5.53
8	663.00	5.97

Roadway Surface: Paved

Roadway Top Width: 30.00 ft

Table HY8- 19 - Summary of Culvert Flows at Crossing: OO_4B_Existing_High_Tide

Headwater Elevation (ft)	Total Discharge (cfs)	OO_04 Existing (Existing High Tide Model) Discharge (cfs)	Roadway Discharge (cfs)	Iterations
2.35	50.00	5.32	44.45	12
2.45	95.00	5.85	88.74	6
2.54	140.00	6.24	133.37	5
2.61	185.00	6.56	177.80	4
2.68	230.00	6.83	222.83	4
2.71	250.00	6.94	242.42	3
2.80	320.00	7.29	312.60	4
2.85	365.00	7.49	357.21	3
2.90	410.00	7.67	402.20	3
2.95	455.00	7.83	447.13	3
2.99	500.00	7.99	491.84	2
2.00	3.05	3.05	0.00	Overtopping

Table HY8- 20 - Culvert Summary Table: OO_04 Existing (Existing High Tide Model)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	5.32	2.35	1.353	3.546	4-FFf	1.500	0.888	1.500	3.830	3.011	0.000
95.00	5.85	2.45	1.440	3.653	4-FFf	1.500	0.931	1.500	3.830	3.310	0.000
140.00	6.24	2.54	1.505	3.739	4-FFf	1.500	0.962	1.500	3.830	3.532	0.000
185.00	6.56	2.61	1.560	3.813	4-FFf	1.500	0.987	1.500	3.830	3.711	0.000
230.00	6.83	2.68	1.608	3.881	4-FFf	1.500	1.009	1.500	3.830	3.868	0.000
250.00	6.94	2.71	1.627	3.908	4-FFf	1.500	1.018	1.500	3.830	3.929	0.000
320.00	7.29	2.80	1.690	3.998	4-FFf	1.500	1.045	1.500	3.830	4.126	0.000
365.00	7.49	2.85	1.726	4.051	4-FFf	1.500	1.059	1.500	3.830	4.236	0.000
410.00	7.67	2.90	1.761	4.101	4-FFf	1.500	1.071	1.500	3.830	4.339	0.000
455.00	7.83	2.95	1.793	4.148	4-FFf	1.500	1.081	1.500	3.830	4.434	0.000
500.00	7.99	2.99	1.824	4.193	4-FFf	1.500	1.092	1.500	3.830	4.522	0.000

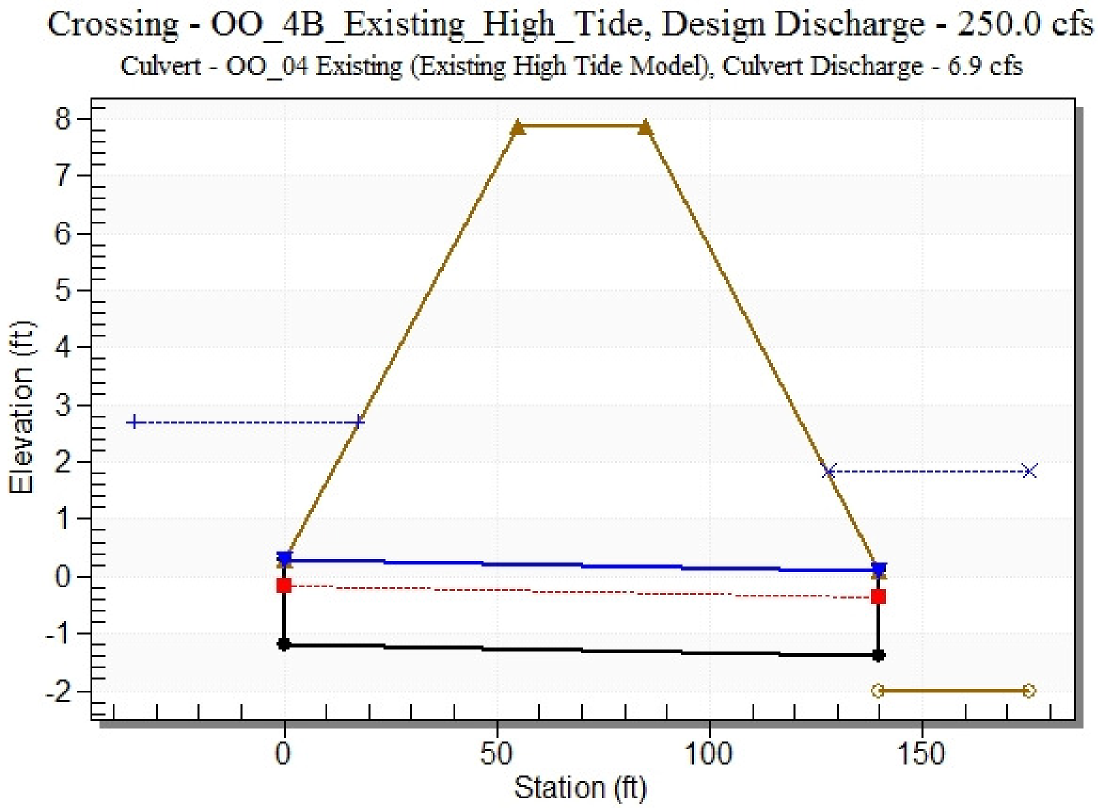
****

Inlet Elevation (invert): -1.20 ft, Outlet Elevation (invert): -1.40 ft

Culvert Length: 140.00 ft, Culvert Slope: 0.0014

****

Water Surface Profile Plot for Culvert: OO_04 Existing (Existing High Tide Model)



Site Data - OO_04 Existing (Existing High Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.20 ft

Outlet Station: 140.00 ft

Outlet Elevation: -1.40 ft

Number of Barrels: 1

Culvert Data Summary - OO_04 Existing (Existing High Tide Model)

Barrel Shape: Circular

Barrel Diameter: 1.50 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: NONE

Table HY8- 21 - Downstream Channel Rating Curve (Crossing: OO_4B_Existing_High_Tide)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
50.00	1.83	3.83
95.00	1.83	3.83
140.00	1.83	3.83
185.00	1.83	3.83
230.00	1.83	3.83
250.00	1.83	3.83
320.00	1.83	3.83
365.00	1.83	3.83
410.00	1.83	3.83
455.00	1.83	3.83
500.00	1.83	3.83

Tailwater Channel Data - OO_4B_Existing_High_Tide

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 1.83 ft

Roadway Data for Crossing: OO_4B_Existing_High_Tide

Roadway Profile Shape: Irregular Roadway Shape (coordinates)

Irregular Roadway Cross-Section:

Coord No.	Station (ft)	Elevation (ft)
0	0.00	7.88
1	98.00	5.89
2	157.00	3.84
3	229.00	3.20
4	393.00	2.00
5	537.00	2.32
6	601.00	3.88
7	636.00	5.53
8	663.00	5.97

Roadway Surface: Paved

Roadway Top Width: 30.00 ft

Concept Design OO_09B

1 EXISTING SITE DESCRIPTION

The questionnaire responses indicate that water ponds yearly on River Road approximately 300 feet west of Cerise Lane and runs onto adjacent properties. There is a 30-inch diameter concrete culvert at this location that drains a 1.3-square mile watershed. The existing culvert is adjacent to the residence of 32522 River Road and several trees. Figure 1 and the photographs show existing conditions.

Debris upstream of existing River Road culvert approximately 300 feet west of the Cerise Lane

Culvert at the Indian River Bay downstream of River Road

During the first field investigation (September 2014), ponding water was observed on the roadway, and debris was observed on the culvert crossing River Road. Tidal water was also observed flowing upstream from the culvert. A catch basin connected to the culvert had standing water. During the second field investigation (February 2015), the culvert was in good condition.

Under existing conditions, runoff from the watershed drains slowly through the existing culvert and often floods River Road. High tidal elevations exacerbate this problem by raising water levels in the marsh to the north. The areas south (i.e., downstream) of River Road near the existing culvert are at a higher elevation than the road. When the culvert cannot convey the required flow, River Road, Cerise Lane, and part of nearby properties flood. There is approximately 570 feet of bulkhead with a top elevation ranging from 2.5 feet to 3.2 feet North American Vertical Datum of 1988 (NAVD88) northeast of the existing culvert.

Figure 1: OO_09 Existing Site Conditions

2 PROPOSED IMPROVEMENT

Existing surcharging catch basin between 32530 and 32522 River Road during high tide

The proposed design at site OO_09 is to raise the existing 570 feet of bulkhead (as needed to reach an elevation of 3.4 feet NAVD88) and then extending it by installing approximately 630 feet of bulkhead north of the roadway. In addition, we propose to increase conveyance by installing three 36-inch diameter culverts under River Road. Figure 2 displays the proposed system. This system would be sized to convey the 25-year storm event (in accordance with the Delaware Department of Transportation [DelDOT] Road Design Manual).

Currently, the high ground south of River Road controls storage within the marsh, but begins to be overtopped at an elevation of approximately 3.5 feet NAVD88, flooding River Road (approximately 2 feet NAVD88) and nearby properties yearly. By raising the existing bulkhead to an elevation of 3.4 feet NAVD88, extending the bulkhead to the north at an elevation of 3.4 feet NAVD88, and extending the bulkhead to the west at an elevation of 3.2 feet NAVD88, flooding on River Road would be reduced for low-probability events. The proposed bulkhead should be composed of impervious material (e.g., concrete), and would be 1 to 2 feet above the existing ground surface elevation. To upgrade the existing wooden bulkhead sheet piles, wood with backfill, or lightweight structural practices should be considered. Concrete is not be practical to upgrade the existing bulkhead, and should only be considered if the existing bulkhead is removed and replaced. The proposed bulkhead to the west of the existing culvert is proposed at an elevation of 3.2 feet NAVD88 instead of 3.4 feet NAVD88 so overflow would be concentrated to River Road and would then spread to the areas that would have been flooded at the existing conditions. This would prevent adverse effects to any of the residences upstream of River Road adjacent to the proposed bulkhead.

The proposed bulkhead would be at a lower elevation than the existing high ground downstream of River Road, limiting negative impacts upstream. Based on preliminary calculations, the backwater elevations are anticipated to decrease for all storm event return periods. The bulkhead would have over 1 foot of freeboard above the computed depth of a storm occurring during the average tide. The bulkhead could be installed at a lower elevation if the conveyance capacity of the system is increased, or if a lower design storm is used.

Location of proposed culverts under the driveway of 32548 River Road

Because the existing culvert is so close to adjacent properties and trees, it is recommended that the proposed culverts be installed under the driveway of 32548 River Road. Based on the existing elevation of River Road, it appears that a 36-inch pipe diameter is the largest that could pass under the road, although a detailed survey will be required for verification. Three 36-inch diameter culverts are proposed to convey the 25-year storm event for high-tide conditions. A headwall is recommended upstream and downstream of the proposed culverts. Backwater control check valves (e.g., Tideflex CheckMate inline check valve) are recommended either at the proposed junction box or at the downstream end of the proposed culverts.

Based on site investigations, it appears that the existing culvert crossing River Road has ample capacity, and from visual inspection it does not appear to be damaged. Further investigation of the pipe will be required, and slip lining will be considered if the pipe is in poor condition. Removing and replacing the existing culvert is not proposed because the pipe is near a residential structure and trees. A headwall and tide gate (e.g., the Waterman self-regulating tide gate) are recommended downstream of the existing culvert to allow saltwater flow into the marsh during low and average tides, while preventing flow during high tide. By providing backwater control check valves for the proposed culverts and a tide gate for the existing culvert, the maximum flow rate out of the marsh would be higher than the flow rate into the marsh. It is recommended that the existing catch basin be converted to a junction box to prevent surcharging from marsh backwater.

Three catch basins are proposed at low areas in the project area. Storm drain pipe would connect the eastern catch basins to the proposed junction box, and the western catch basin to the existing catch basin that is proposed to be converted to a junction box. The minimum DelDOT-recommended storm drain pipe size of 15 inches is recommended. Backwater control check valves are recommended in the storm drain pipe to prevent surcharge onto River Road when marsh levels are elevated. The inline check valves could be placed at either the upstream or downstream end of the pipes, depending on preferred maintenance locations.

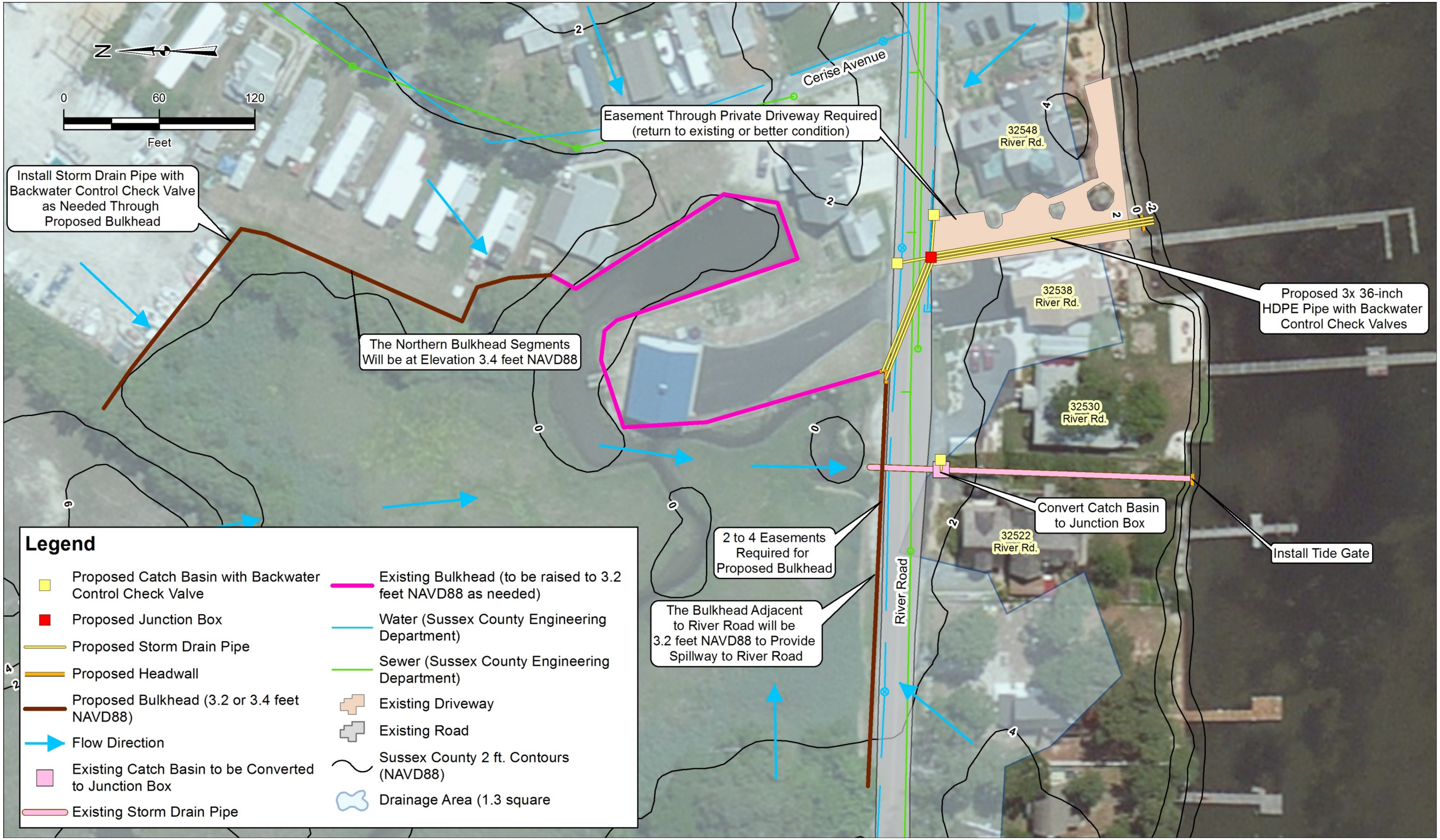


Figure 2: OO_09 Proposed Site Design

3 HYDROLOGIC CALCULATIONS

3.1 HEC-HMS Hydrologic Analysis

A hydrologic analysis for Oak Orchard was performed using HEC-HMS. The methodology and results of this study are discussed in Appendix F. The peak flows into the existing culvert at OO_09 are calculated at Outfall 5. These flows do not account for the flow through the existing culvert or the storage behind the culvert, as these factors will vary with the proposed design. The peak flows to the existing culvert without accounting for culvert hydraulics or storage for the 2-, 10- , 25-, 50-, and 100-year-annual-chance recurrence are approximately 60, 150, 220, 300, and 380 cubic feet per second, respectively. Figure 3 displays the inflow hydrographs calculated in HEC-HMS. These inflow hydrographs are used with storage information to compute hydraulic discharge and calculate the peak flow through the existing and proposed culverts.

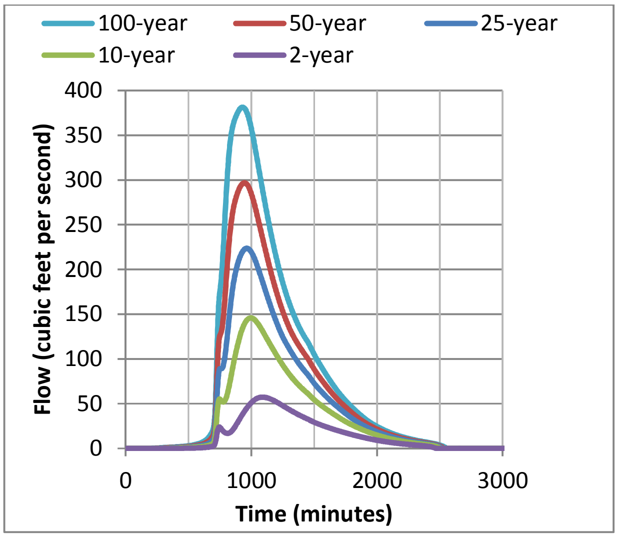


Figure 3: OO_09 Inflow Hydrographs Calculated in HEC-HMS

3.2 Storage Calculations

The marsh upstream of the existing and proposed culvert at OO_09 provides storage for the flow upstream of the existing and proposed culverts crossing River Road. The National Elevation Dataset (NED) 1/9-arc-second (3-meter) raster from the U.S. Geological Survey (USGS) was used to calculate the storage upstream of the existing and proposed culverts (USGS, 2007). The vertical datum for the topographic data is NAVD88. The high point for the proposed design is the proposed bulkhead elevation (minimum elevation of 3.2 feet NAVD88), while the area south of River Road are the high points for the existing conditions.

The elevation raster was converted to a triangulated irregular network over the storage area, and the polygon volume GIS function was used to calculate the volume for various elevations (ESRI, 2012). It was assumed that under existing conditions the marsh level is most likely to be at the average tide elevation (0.5 foot NAVD88), while the proposed marsh elevation is assumed to be the average low tide elevation (-0.83 foot NAVD88) due to the backwater control check valves and the tide gate. A detailed discussion of the tidal elevations is provided in Section 4.1. Figure 4 displays the proposed bulkhead and upstream area with 2-foot contours, and Table 1 shows the storage volumes for the existing and proposed site conditions.

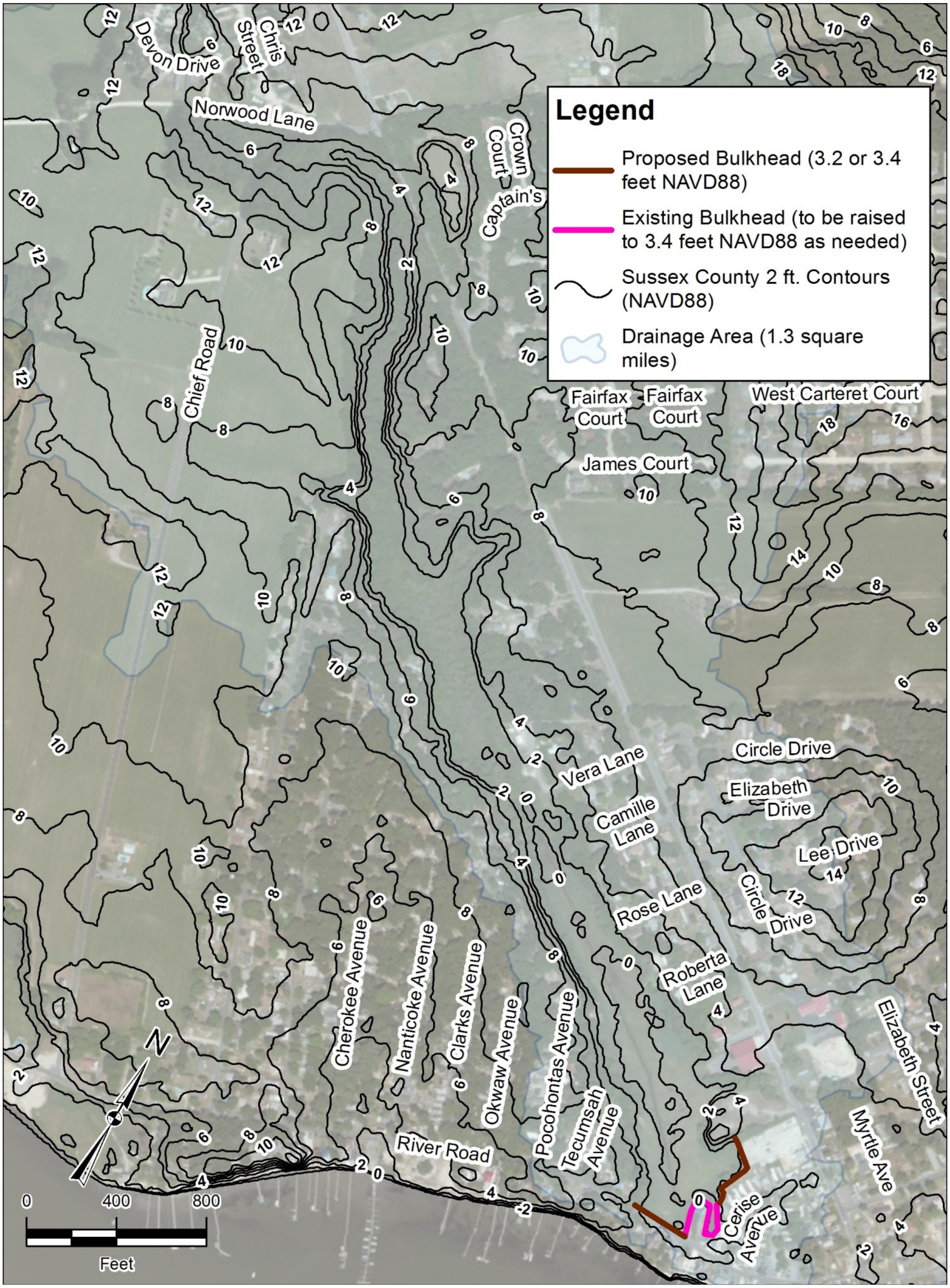


Figure 4: OO_09 Proposed Bulkhead and Upstream Storage Area

Table 1: OO_09 Existing and Proposed Conditions Stage-Storage Relationship

Elevation (feet NAVD88)	Existing Conditions Storage Volume (acre-feet)	Proposed Conditions Storage Volume (acre-feet)
-0.83	0.0	0.0
-0.5	0.0	0.5
0	0.0	4.0
0.5	0.0	12.0
1	7.8	18.1
1.5	18.0	21.6
2	30.2	24.7
2.5	44.7	27.5
3	61.6	30.5
3.5	81.1	34.0[a]
4	103.4	38.1[a]

[a] This volume only includes the storage upstream of the proposed bulkhead, although the areas downstream would be expected to flood once the proposed bulkhead was overtopped.

4 HYDRAULIC CALCULATIONS

4.1 Tidal Tailwater Boundary Conditions

The hydraulic conditions of this site are influenced by the Indian River Bay. The USGS Indian River stream gage at Rosedale Beach (Gage 01484540) is located less than a mile from the Oak Orchard community, and was used to estimate average low tide, high tide, and overall average water surface elevations for the Indian River Bay (USGS, 2012). The gage data were provided in the National Geodetic Vertical Datum of 1929 (NGVD29) and corrected to NAVD88 by adding -0.78 foot to the NGVD29 elevation. According to daily data from 2006 to 2015, the average low tide elevation is -0.83 foot, the average high tide elevation is 1.83 feet, and the overall average elevation is 0.5 foot. The time between low tide and high tide is approximately 6 hours, while the time between initial hydrograph response and peak discharge is over 5 hours at OO_09. The tidal conditions would therefore be expected to vary from low tide to high tide for an actual event, although a single tide elevation simplification is used for this study.

4.2 Existing Hydraulic Calculations

The culvert hydraulic conditions were calculated using HY-8 Culvert Analysis Program version 7.2 (FHWA, 2012). Rating curves (flow versus stage) were calculated for average low tide, average high tide, and overall average water surface elevations for the Indian River Bay. The rating curves are provided in Table 2. The HY-8 output is provided in Section 10 with the 25-year design flow and flows from 50 cubic feet per second to 500 cubic feet per second. HY-8 calculates headwater elevations for only 11 flows at a time, so HY-8 was run with multiple flow ranges to calculate the rating curve provided in Table 2. The HY-8 flow ranges were 0 to 50 cubic feet per second, 50 to 130 cubic feet per second, 130 to 170 cubic feet per second, 170 to 250 cubic feet per second, and 250 to 1000 cubic feet per second.

Table 2: Existing Conditions Rating Curves for Varying Indian River Bay Tailwater Boundary Conditions

Discharge, cubic feet per second	Water Surface Elevation Upstream of Existing River Road for Varying Indian River Bay Tailwater Conditions (feet NAVD88)		
	Tailwater = 0.5 (feet NAVD88)	Tailwater = 1.83 (feet NAVD88)	Tailwater = -0.83 (feet NAVD88)
0	0.5	1.83	-0.83
5	0.54	1.87	-0.61
10	0.67	2	-0.17
15	0.89	2.22	0.27
20	1.2	2.53	0.69
25	1.59	2.92	1.15
30	2.07	3.4	1.75
35	2.64	3.61	2.41
40	3.3	3.65	3.15
45	3.6	3.68	3.59
50	3.65	3.71	3.64
58	3.7	3.73	3.69
66	3.73	3.76	3.73
74	3.75	3.78	3.75
82	3.77	3.79	3.77
90	3.79	3.81	3.79
98	3.8	3.82	3.8
106	3.82	3.83	3.82

Discharge, cubic feet per second	Water Surface Elevation Upstream of Existing River Road for Varying Indian River Bay Tailwater Conditions (feet NAVD88)		
	Tailwater = 0.5 (feet NAVD88)	Tailwater = 1.83 (feet NAVD88)	Tailwater = -0.83 (feet NAVD88)
114	3.83	3.85	3.83
122	3.84	3.86	3.84
130	3.86	3.87	3.85
134	3.86	3.87	3.86
138	3.87	3.88	3.87
142	3.87	3.88	3.87
146	3.88	3.89	3.87
150	3.88	3.89	3.88
154	3.89	3.9	3.88
158	3.89	3.9	3.89
162	3.89	3.91	3.89
166	3.9	3.91	3.9
170	3.9	3.91	3.9
178	3.91	3.92	3.91
186	3.92	3.93	3.92
194	3.93	3.94	3.93
202	3.94	3.94	3.93
210	3.94	3.95	3.94
218	3.95	3.96	3.95
226	3.95	3.96	3.95
234	3.96	3.97	3.96
242	3.97	3.97	3.97
250	3.97	3.98	3.97
325	4.03	4.03	4.02
400	4.07	4.08	4.07
475	4.11	4.12	4.11
550	4.15	4.15	4.15
625	4.18	4.19	4.18
700	4.21	4.22	4.21
775	4.24	4.25	4.24
850	4.27	4.28	4.27
925	4.3	4.3	4.3
1000	4.33	4.33	4.33

The inflow hydrographs from HEC-HMS, the existing stage-storage curves, and the existing conditions rating curves calculated using HY-8 were used to compute outflow hydrographs. The hydrograph routing was performed using the Hydraulic Toolbox version 4.2 (FHWA, 2014). The road begins to be overtopped at an elevation of approximately 2 feet NAVD88, and the existing high ground south of River Road begins to be overtopped at an elevation of approximately 3.5 feet NAVD88. Table 3 and Table 4 summarize the results of the hydrograph routing. The road is not expected to be overtopped for the 2-year storm event under existing conditions with the average Indian River Bay water surface elevation (0.5 foot NAVD88), but would be overtopped if there were an extended average high-tide (1.83 feet NAVD88). These results are consistent with the questionnaires and conversations with residents in the field that indicate the road typically floods yearly. Figure 5 displays the inflow and outflow hydrographs for the 2-year storm event with the average Indian River Bay water surface elevation (0.5 foot NAVD88). Only the 2-year storm event is displayed in Table 3 because it is the only storm event frequency that would not overtop the roadway under existing conditions.

Table 3 Existing Conditions Peak Flow and Peak Water Surface Elevation Calculated from Hydrograph Routing for 2-year Storm Event with Varying Tailwater Conditions

Tide Description	Tide Elevation (feet NAVD88)	Existing Conditions	
		Peak Flow (cubic feet per second)	Peak Stage (feet NAVD88)
Average Low Tide	-0.83	28	1.5
Average Tide	0.5	26	1.7
Average High Tide	1.83	17	2.31[a]

[a] = Expected to overtop existing roadway

Table 4: Existing Conditions Peak Flow and Peak Water Surface Elevation Calculated from Hydrograph Routing for Average Tide Tailwater Conditions (0.5 foot NAVD88)

Storm Event	Existing Conditions		FEMA Stillwater Elevation[b] (feet NAVD88)
	Peak Flow (cubic feet per second)	Peak Stage (feet NAVD88)	
2-year	30	1.7	Not Provided
10-year	40	3.4[a]	3.8
25-year	130	3.9[a]	Not Provided
50-year	210	3.9[a]	5.6
100-year	310	4.0[a]	6.7

[a] = Expected to overtop existing roadway
[b] = The stillwater elevations for the 10-, 50-, and 100-year storm events are from the Sussex County 2005 Flood Insurance Study (FIS) Report (FEMA, 2005), although the flood maps indicate that the 100 year base flood elevations range from 7 to 9 feet over the study area

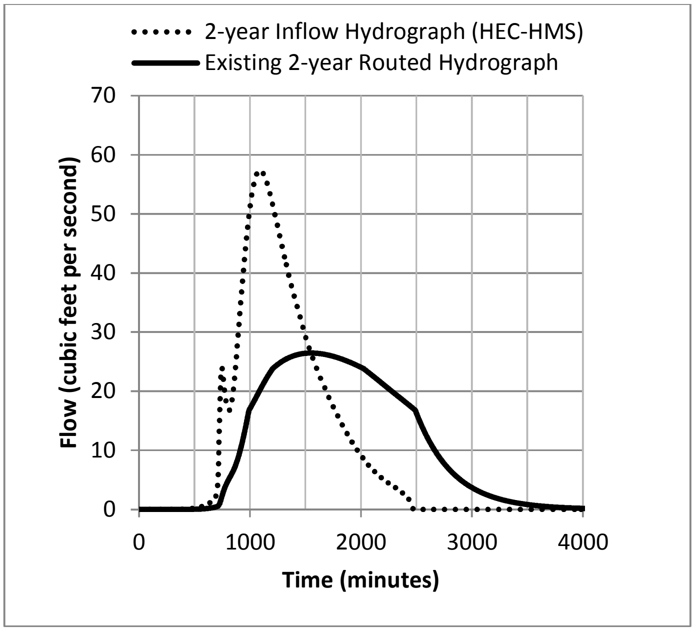


Figure 5: Existing Conditions 2-year Inflow and Routed Hydrograph (0.5 foot NAVD88 tailwater boundary condition)

4.3 Proposed Hydraulic Calculations

The proposed hydraulic conditions were calculated using the same method used for the existing hydraulic conditions. HY-8 was used to compute rating curves for the proposed culvert and bulkhead conditions. An iterative approach was used to calculate the required pipe sizes and bulkhead elevation based on the existing site constraints. The final rating curves (flow versus stage) were calculated for average low tide, average high tide, and overall average water surface elevations for the Indian River Bay. The rating curves are shown in Table 5.

Table 5: Proposed Conditions Rating Curves for Varying Indian River Bay Tailwater Boundary Conditions

Discharge, cubic feet per second	Water Surface Elevation Upstream of Proposed Bulkhead for Varying Indian River Bay Tailwater Conditions (feet NAVD88)		
	Tailwater = 0.5 (feet NAVD88)	Tailwater = 1.83 (feet NAVD88)	Tailwater = -0.83 (feet NAVD88)
0	0.5	1.83	-0.83
5	0.5	1.83	-0.71
10	0.51	1.84	-0.70
15	0.51	1.84	-0.70
20	0.52	1.85	-0.70
25	0.53	1.87	-0.67
30	0.55	1.88	-0.65
35	0.56	1.9	-0.56
40	0.58	1.92	-0.46
45	0.6	1.95	-0.37
50	0.62	1.98	-0.27
58	0.67	2.03	-0.12
66	0.71	2.08	0.02
74	0.77	2.15	0.16
82	0.82	2.22	0.28
90	0.89	2.3	0.41

Discharge, cubic feet per second	Water Surface Elevation Upstream of Proposed Bulkhead for Varying Indian River Bay Tailwater Conditions (feet NAVD88)		
	Tailwater = 0.5 (feet NAVD88)	Tailwater = 1.83 (feet NAVD88)	Tailwater = -0.83 (feet NAVD88)
98	0.95	2.39	0.52
106	1.16	2.49	0.64
114	1.26	2.59	0.76
122	1.37	2.7	0.87
130	1.49	2.82	0.99
134	1.55	2.88	1.05
138	1.61	2.94	1.12
142	1.68	3	1.18
146	1.75	3.07	1.25
150	1.82	3.14	1.31
154	1.89	3.2	1.38
158	1.96	3.23	1.45
162	2.03	3.24	1.52
166	2.11	3.25	1.59
170	2.19	3.27	1.67
178	2.35	3.29	1.82
186	2.52	3.31	1.98
194	2.7	3.33	2.15
202	2.88	3.34	2.32
210	3.07	3.36	2.5
218	3.22	3.37	2.69
226	3.25	3.39	2.88
234	3.28	3.4	3.1
242	3.3	3.42	3.23
250	3.32	3.43	3.26
325	3.45	3.54	3.42
400	3.56	3.63	3.54
475	3.66	3.72	3.63
550	3.74	3.8	3.72
625	3.82	3.88	3.8
700	3.9	3.95	3.88
775	3.97	4.02	3.95
850	4.04	4.08	4.02
925	4.1	4.15	4.09
1000	4.17	4.21	4.15

The inflow hydrographs from HEC-HMS, the proposed conditions stage-storage curves, and the proposed conditions rating curves calculated using HY-8 were used to compute an outflow hydrograph. The hydrograph routing was performed using the Hydraulic Toolbox version 4.2 (FHWA, 2014). The proposed bulkhead would not be overtopped until an elevation of 3.2 feet NAVD88. Table 6 and Table 7 summarize the results of the hydrograph routing. The proposed bulkhead is expected to be overtopped for the 100-year storm event, but allow the 25-year and 50-year storm events to be conveyed without impacting River Road, even if it occurred entirely during the average high tide (which is unrealistic given the duration of tides and the anticipated duration of the storm event as discussed in Section 4.1). The 25-year storm during average high tide dictated the bulkhead height for the design.

Figure 6 displays the inflow and proposed outflow hydrographs for the 25-year storm event with the average Indian River Bay water surface elevation (0.5 foot NAVD88).

Table 6: Proposed Conditions Peak Flow and Peak Water Surface Elevation Calculated from Hydrograph Routing for 25-year Storm Event for Varying Tailwater Conditions

Tide Description	Tide Elevation (feet NAVD88)	Proposed Conditions	
		Peak Flow (cubic feet per second)	Peak Stage (feet NAVD88)
Average Low Tide	-0.83	172	1.7
Average Tide	0.5	166	2.1
Average High Tide	1.83	155	3.2

Table 7: Proposed Conditions Peak Flow and Peak Water Surface Elevation Calculated from Hydrograph Routing for Average Tide Tailwater Conditions (0.5 foot NAVD88)

Storm Event	Proposed Conditions		FEMA Stillwater Elevation[b] (feet NAVD88)
	Peak Flow (cubic feet per second)	Peak Stage (feet NAVD88)	
2-year	60	0.7	Not Provided
10-year	120	1.4	3.8
25-year	170	2.1	Not Provided
50-year	200	2.8	5.6
100-year	300	3.4[a]	6.7

[a] = Expected to overtop proposed bulkhead
[b] = The stillwater elevations for the 10-, 50-, and 100-year storm events are from the Sussex County 2005 FIS Report (FEMA, 2005), although the flood maps indicate that the 100-year base flood elevations range from 7 to 9 feet over the study area

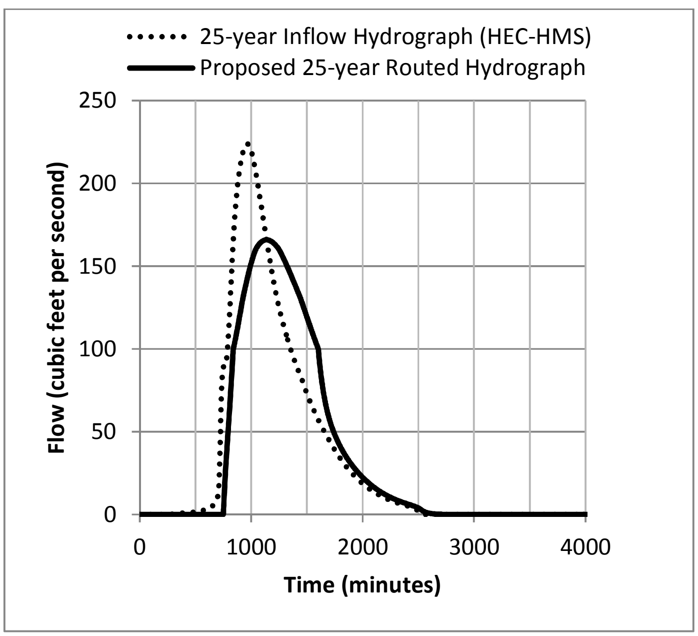


Figure 6: Proposed Conditions 25-year Inflow and Routed Hydrograph (0.5 foot NAVD88 tailwater boundary condition)

5 IMPROVEMENTS AND BENEFITS

The proposed design would reduce the frequency and duration of water ponding on and adjacent to this section of River Road by increasing conveyance capacity and redistributing storage.

The existing storage area (i.e., ponding area) when the flow exceeds the existing pipe capacity includes River Road and the nearby properties, and there is no mechanism to drain it. Installing the bulkhead will remove River Road and nearby properties from the storage area for the 10-, 25-, and 50-year storm events. The three catch basins will allow this area to drain when the tide or the marsh level falls below the road elevation. Table 4 and Table 7 provide the existing conditions and proposed water surface elevations along with the stillwater elevations from the FEMA 2005 FIS Report.

It is recommended that a non-electronic, self-regulating tide gate (e.g., the Waterman self-regulating tide gate) be installed at this location to convey saltwater into the marsh for biological habitat during low and average tides, while preventing flow during high tide. A headwall would be required to install a self-regulating tide gate. By installing backwater control check valves for the three proposed 36-inch-diameter pipes and a tide gate for the existing 30-inch-diameter pipe, the maximum flow rate from the marsh to the Indian River Bay would be substantially higher than the flow rate from the Indian River Bay back into the marsh.

6 FEASIBILITY ASSESSMENT

Soil and Groundwater: The soils at the proposed design site are primarily hydrologic group A, which are well drained and primarily composed of sand (NRCS, 2009). The marsh area is primarily hydrologic group D soils, which are very poorly drained with high clay content. Groundwater data from the Delaware Geologic Survey suggest that the water table is approximately 1 foot below the ground surface. When the water table is high, standing water is expected in the marsh area whether or not there is a direct connection to the Indian River Bay.

Construction Access: This site is accessible from River Road, although a portion of the culvert and the bulkhead would be constructed on private property. According to the Sussex County, Delaware GIS parcel database, approximately 3 to 5 easements will be required (Sussex County Assessment Office, 2008). This will need to be verified during final design. There is limited space north and south of River Road, so temporary easements may be required to park construction equipment.

Maintenance Considerations: Routine maintenance of culverts, backwater control check valves, and the tide gate will be required to sustain culvert capacity. Maintenance would include periodically removing sediment and debris from inlets, catch basins, backwater control check valves, and the tide gate. It is critical that the backwater control check valves and tide gate be maintained regularly to ensure that this systems functions as intended.

Utility Conflicts: The Sussex County Engineering Department provided as-built plans for the Sewer installation at Oak Orchard. The proposed culvert would cross a 6-inch water line at an unknown elevation, an 8-inch sewer force main at an unknown elevation, and an 8-inch PVC sewer line with an invert elevation of approximately -2.8 feet NAVD88. The proposed culvert would be installed several inches above the existing sewer line, although a field survey would be required to verify the exact elevation of the existing sewer line and how it would impact the culvert design. Above-ground electric lines are located south of River Road at this location and are not anticipated to impact construction. There may be underground cable lines, which will need to be confirmed during detailed design.

Effectiveness: The proposed design would substantially reduce nuisance flooding from frequent storm events on River Road and the nearby properties. Flooding from large coastal events would still be expected; however, the duration of flooding should be reduced. The effectiveness of the proposed design would be heavily dependent on the routine maintenance of the proposed culverts, check valves, and tide gate. By installing the western bulkhead at a lower elevation than the rest of the bulkhead, an emergency spillway will be created that would convey flow to River Road, where it would be allowed to spread (as it would if the bulkhead were not present). This would prevent additional flooding at the houses north of River Road that are adjacent to the proposed bulkhead.

Environmental Issues: Substantial environmental impacts are expected because construction would occur adjacent or within the existing marsh. There are approximately 5 to 10 trees along the proposed bulkhead, as well as invasive phragmites. The invasive phragmites can be removed, but a detailed survey will be required to design the bulkhead in a way that limits impact on existing trees. It is possible that wetland vegetation may also be affected, although the removal of the phragmites will benefit native vegetation.

Impact on Base Flood Elevations: The stillwater elevations for the for the 10- 50-, and 100-year-annual-chance recurrence are provided in the FEMA 2005 FIS report for Sussex County (Table 4 and Table 7). These values are all greater than the backwater elevations calculated upstream of the proposed bulkhead. Coastal backwater from the Indian River Bay is therefore still expected to be the predominant control for low-probability events. The proposed design is expected to prevent flooding from higher probability precipitation events (2-year to 50-year events) without increasing frequency or duration of flooding from the 100-year event.

7 PLANS AND PERMITTING

Several construction documents and plans would need to be obtained to implement the proposed drainage design, including, but not limited to those described in Table 8.

Table 8: Required Plans and Permitting for Proposed OO_09 Design

Plans/Permits	Permitting Agency	Notes and Potential Difficulties
Wetlands and Subaqueous Lands Permit	DNREC	The proposed culvert and bulkhead will impact the marsh north of the River Road.
Traffic Control Plan	DelDOT	River Road will be impacted while the proposed culverts and catch basins are installed.
Erosion and Sediment Control Plan	County Conservation District	
Utility Construction Permit	DelDOT	Utility impacts are possible for this project due to several sewer and water lines in the area. It may be possible to avoid existing utilities for the proposed layout, or the layout may need to be adjusted to minimize impacts. Care will need to be taken to avoid damaging the existing infrastructure, and all construction will be coordinated with DelDOT and the Sussex County Engineering Department.

8 COST ESTIMATE

Table 9 summarizes the costs associated with this concept design.

Table 9 Estimated Project Costs for OO_09

ITEM	QUANTITY	UNITS	UNIT COST		TOTAL
Excavation	980	CY	$25.00		$24,500
Grading	430	SY	$2.50		$1,075
Bulkhead (Reinforced Concrete)	280	CY	$800.00		$224,000
Upgrade Existing Bulkhead	570	LF	$250.00		$142,500
Asphalt Base	17	TON	$100.00		$1,700
Asphalt Surface	43	TON	$110.00		$4,730
Hydroseeding	40	SY	$0.75		$30
15" Backflow Control Check Valve	3	EA	$3,300.00		$9,900
36" Backflow Control Check Valve	3	EA	$9,449.00		$28,347
30" Self-Regulating Tide Gate	1	EA	$40,000.00		$40,000
Inlet	3	EA	$2,500.00		$7,500
Junction Box	2	EA	$2,500.00		$5,000
Traffic Control Plan	7	DAY	$750.00		$5,250
Endwall (36" pipe)	6	EA	$3,500.00		$21,000
Endwall (30" pipe)	1	EA	$2,500.00		$2,500
15" High Density Polyethylene Pipe	60	LF	$45.00		$2,700
36" High Density Polyethylene Pipe	660	LF	$80.00		$52,800
			Initial Project Costs		$573,532
			Contingency	10%	$57,353
		Erosion and Sediment Control		10%	$57,353
		Base Construction Costs			$688,238
		Mobilization		5%	$34,412
			Subtotal 1		$722,650
		Contingency		15%	$108,398
			Subtotal 2		$831,048
		Engineering			$120,000
				Total	$951,048

CY = cubic yard
EA = each
LF = linear foot
SY = square yard

9 REFERENCES

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10 HY-8 REPORT

Table HY8-1 - Summary of Culvert Flows at Crossing: OO_9B_Proposed_Average_Tide

Headwater Elevation (ft)	Total Discharge (cfs)	OO_09 Proposed (Proposed Average Tide Model) Discharge (cfs)	Existing 30" Culvert (Proposed Average Tide Model) Discharge (cfs)	Roadway Discharge (cfs)	Iterations
0.62	50.00	41.97	8.34	0.00	10
0.93	95.00	79.24	15.70	0.00	8
1.65	140.00	114.42	25.61	0.00	3
2.11	166.00	135.65	30.37	0.00	4
3.26	230.00	177.60	39.76	11.89	12
3.37	275.00	180.95	40.52	53.03	6
3.45	320.00	183.43	41.07	95.16	5
3.51	365.00	185.52	41.53	137.43	4
3.58	410.00	187.41	41.96	180.40	4
3.63	455.00	189.10	42.34	222.78	3
3.69	500.00	190.71	42.70	266.02	3
3.20	214.88	175.57	39.31	0.00	Overtopping

Table HY8-2 - Culvert Summary Table: OO_09 Proposed (Proposed Average Tide Model)

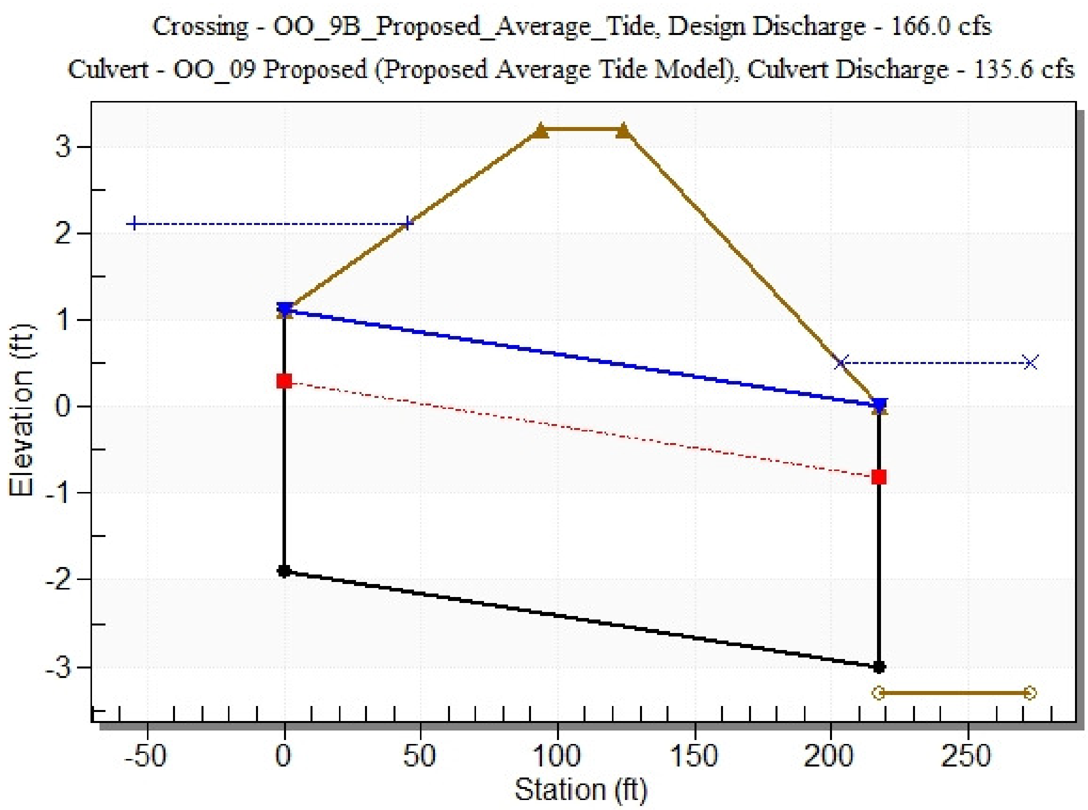
Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	41.97	0.62	1.654	2.522	1-S1f	1.062	1.190	3.000	3.800	1.979	0.000
95.00	79.24	0.93	2.410	2.832	1-S1f	1.524	1.654	3.000	3.800	3.737	0.000
140.00	114.42	1.65	3.036	3.546	4-FFf	1.926	2.004	3.000	3.800	5.396	0.000
166.00	135.65	2.11	3.460	4.011	4-FFf	2.190	2.183	3.000	3.800	6.397	0.000
230.00	177.60	3.26	4.493	5.162	4-FFf	3.000	2.473	3.000	3.800	8.375	0.000
275.00	180.95	3.37	4.588	5.267	4-FFf	3.000	2.491	3.000	3.800	8.533	0.000
320.00	183.43	3.45	4.660	5.347	4-FFf	3.000	2.504	3.000	3.800	8.650	0.000
365.00	185.52	3.51	4.721	5.414	4-FFf	3.000	2.515	3.000	3.800	8.749	0.000
410.00	187.41	3.58	4.776	5.476	4-FFf	3.000	2.525	3.000	3.800	8.838	0.000
455.00	189.10	3.63	4.827	5.531	4-FFf	3.000	2.534	3.000	3.800	8.917	0.000
500.00	190.71	3.69	4.875	5.585	4-FFf	3.000	2.543	3.000	3.800	8.994	0.000

Inlet Elevation (invert): -1.90 ft, Outlet Elevation (invert): -3.00 ft

Culvert Length: 218.00 ft, Culvert Slope: 0.0050

****

Water Surface Profile Plot for Culvert: OO_09 Proposed (Proposed Average Tide Model)



Site Data - OO_09 Proposed (Proposed Average Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.90 ft

Outlet Station: 218.00 ft

Outlet Elevation: -3.00 ft

Number of Barrels: 3

Culvert Data Summary - OO_09 Proposed (Proposed Average Tide Model)

Barrel Shape: Circular

Barrel Diameter: 3.00 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Beveled Edge (1:1)

Inlet Depression: NONE

Table HY8-3 - Culvert Summary Table: Existing 30" Culvert (Proposed Average Tide Model)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	8.34	0.62	1.350	2.502	4-FFf	1.502	0.957	2.500	3.800	1.699	0.000
95.00	15.70	0.93	1.995	2.811	4-FFf	2.500	1.332	2.500	3.800	3.198	0.000
140.00	25.61	1.65	2.772	3.526	4-FFf	2.500	1.723	2.500	3.800	5.218	0.000
166.00	30.37	2.11	3.202	3.991	4-FFf	2.500	1.871	2.500	3.800	6.186	0.000
230.00	39.76	3.26	4.261	5.143	4-FFf	2.500	2.108	2.500	3.800	8.100	0.000
275.00	40.52	3.37	4.360	5.248	4-FFf	2.500	2.124	2.500	3.800	8.254	0.000
320.00	41.07	3.45	4.434	5.327	4-FFf	2.500	2.135	2.500	3.800	8.367	0.000
365.00	41.53	3.51	4.496	5.394	4-FFf	2.500	2.145	2.500	3.800	8.461	0.000
410.00	41.96	3.58	4.554	5.456	4-FFf	2.500	2.154	2.500	3.800	8.547	0.000
455.00	42.34	3.63	4.607	5.512	4-FFf	2.500	2.162	2.500	3.800	8.624	0.000
500.00	42.70	3.69	4.657	5.565	4-FFf	2.500	2.170	2.500	3.800	8.698	0.000

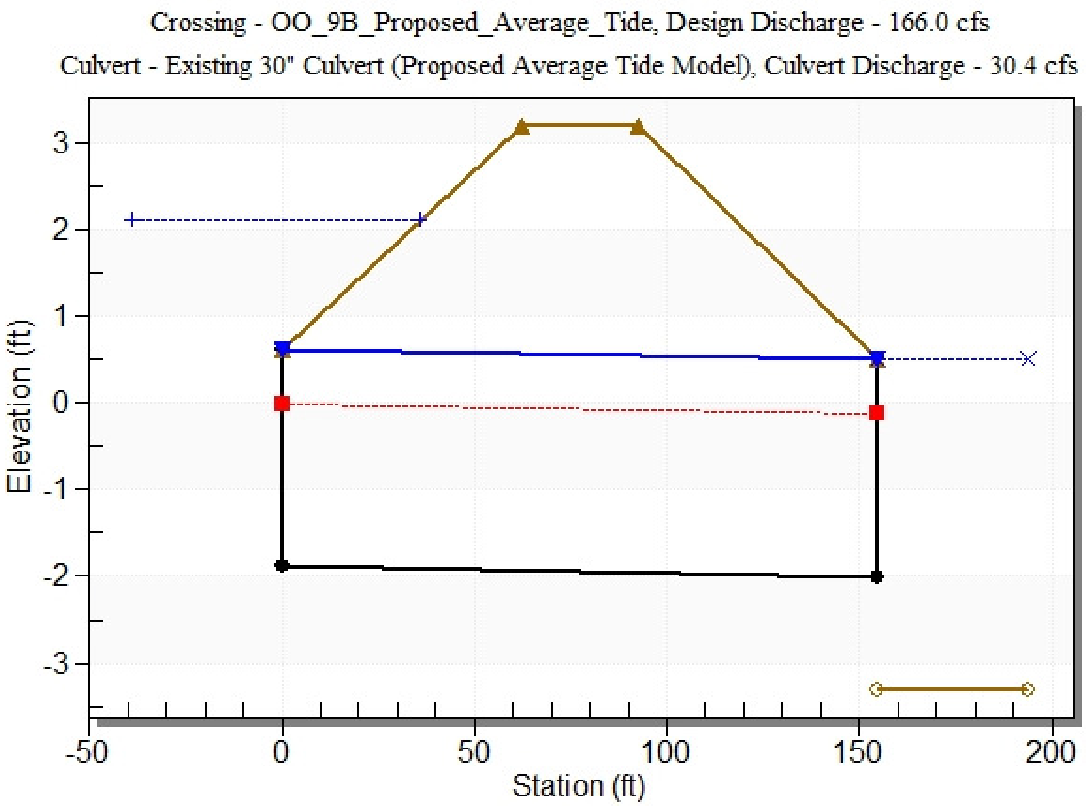
****

Inlet Elevation (invert): -1.88 ft, Outlet Elevation (invert): -2.00 ft

Culvert Length: 155.00 ft, Culvert Slope: 0.0008

****

Water Surface Profile Plot for Culvert: Existing 30" Culvert (Proposed Average Tide Model)



Site Data - Existing 30" Culvert (Proposed Average Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.88 ft

Outlet Station: 155.00 ft

Outlet Elevation: -2.00 ft

Number of Barrels: 1

Culvert Data Summary - Existing 30" Culvert (Proposed Average Tide Model)

Barrel Shape: Circular

Barrel Diameter: 2.50 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: NONE

Table HY8-4 - Downstream Channel Rating Curve (Crossing: OO_9B_Proposed_Average_Tide)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
50.00	0.50	3.80
95.00	0.50	3.80
140.00	0.50	3.80
166.00	0.50	3.80
230.00	0.50	3.80
275.00	0.50	3.80
320.00	0.50	3.80
365.00	0.50	3.80
410.00	0.50	3.80
455.00	0.50	3.80
500.00	0.50	3.80

Tailwater Channel Data - OO_9B_Proposed_Average_Tide

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 0.50 ft

Roadway Data for Crossing: OO_9B_Proposed_Average_Tide

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 260.00 ft

Crest Elevation: 3.20 ft

Roadway Surface: Paved

Roadway Top Width: 30.00 ft

Table HY8-5 - Summary of Culvert Flows at Crossing: OO_9B_Proposed_Low_Tide

Headwater Elevation (ft)	Total Discharge (cfs)	OO_09_Proposed (Proposed Low Tide Model) Discharge (cfs)	Existing 30" Culvert (Proposed Low Tide Model) Discharge (cfs)	Roadway Discharge (cfs)	Iterations
-0.27	50.00	41.09	8.99	0.00	8
0.48	95.00	77.54	17.46	0.00	4
1.15	140.00	115.02	25.00	0.00	4
1.71	172.00	142.35	29.68	0.00	4
2.98	230.00	190.70	38.90	0.00	34
3.32	275.00	199.66	41.16	33.45	10
3.41	320.00	201.98	41.73	75.65	5
3.49	365.00	203.88	42.20	118.14	4
3.55	410.00	205.58	42.61	161.47	4
3.61	455.00	207.12	42.98	204.75	4
3.66	500.00	208.53	43.33	247.52	3
3.20	236.74	196.38	40.36	0.00	Overtopping

Table HY8-6 - Culvert Summary Table: OO_09_Proposed (Proposed Low Tide Model)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	41.09	-0.27	1.632	0.0*	1-S2n	1.050	1.175	1.065	2.470	6.074	0.000
95.00	77.54	0.48	2.380	0.0*	1-S2n	1.504	1.635	1.505	2.470	7.284	0.000
140.00	115.02	1.15	3.048	0.0*	5-S2n	1.934	2.010	1.939	2.470	7.948	0.000
172.00	142.35	1.71	3.607	3.547	2-M2c	2.283	2.234	2.243	2.470	8.388	0.000
230.00	190.70	2.98	4.875	4.877	7-M2c	3.000	2.543	2.561	2.470	9.891	0.000
275.00	199.66	3.32	5.151	5.224	7-M2c	3.000	2.591	2.610	2.470	10.194	0.000
320.00	201.98	3.41	5.225	5.312	7-M2c	3.000	2.603	2.622	2.470	10.275	0.000
365.00	203.88	3.49	5.286	5.385	7-M2c	3.000	2.613	2.632	2.470	10.341	0.000
410.00	205.58	3.55	5.341	5.450	7-M2c	3.000	2.622	2.636	2.470	10.413	0.000
455.00	207.12	3.61	5.391	5.509	7-M2c	3.000	2.631	2.643	2.470	10.470	0.000
500.00	208.53	3.66	5.437	5.563	7-M2c	3.000	2.638	2.649	2.470	10.522	0.000

* theoretical depth is impractical. Depth reported is corrected.

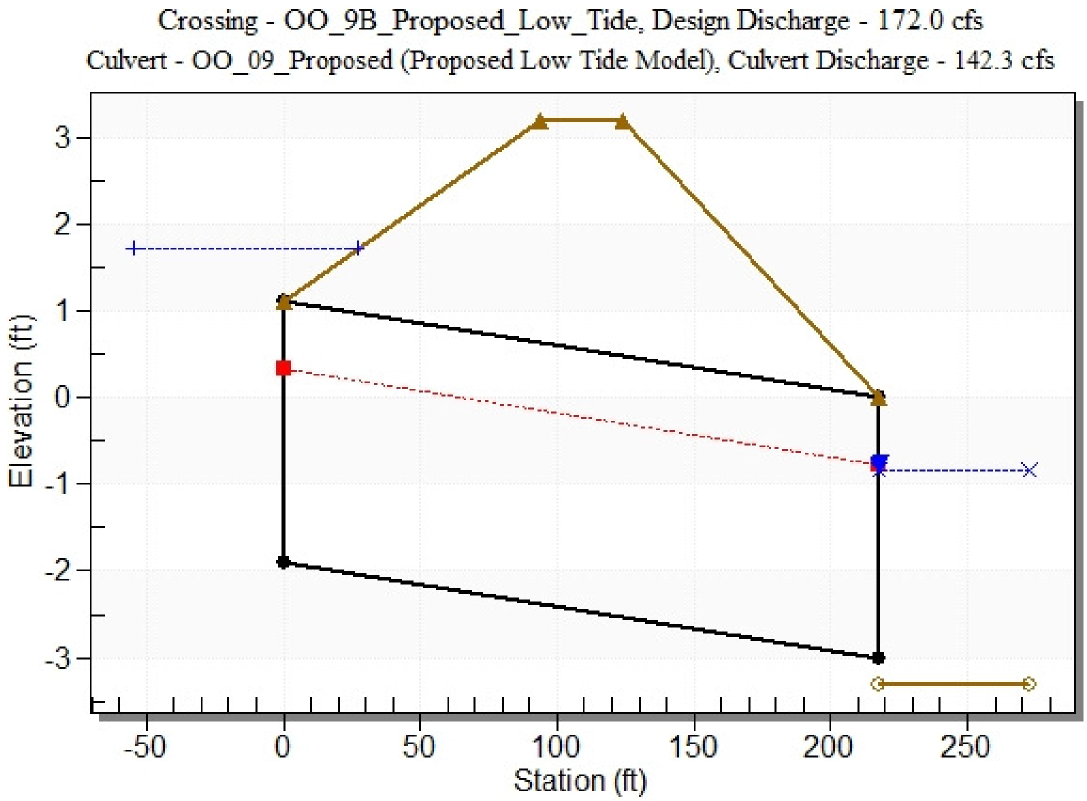
****

Inlet Elevation (invert): -1.90 ft, Outlet Elevation (invert): -3.00 ft

Culvert Length: 218.00 ft, Culvert Slope: 0.0050

****

Water Surface Profile Plot for Culvert: OO_09_Proposed (Proposed Low Tide Model)



Site Data - OO_09_Proposed (Proposed Low Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.90 ft

Outlet Station: 218.00 ft

Outlet Elevation: -3.00 ft

Number of Barrels: 3

Culvert Data Summary - OO_09_Proposed (Proposed Low Tide Model)

Barrel Shape: Circular

Barrel Diameter: 3.00 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Beveled Edge (1:1)

Inlet Depression: NONE

Table HY8-7 - Culvert Summary Table: Existing 30" Culvert (Proposed Low Tide Model)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	8.99	-0.27	1.415	1.613	3-M2t	1.581	0.999	1.170	2.470	3.986	0.000
95.00	17.46	0.48	2.131	2.359	2-M2c	2.500	1.409	1.414	2.470	6.098	0.000
140.00	25.00	1.15	2.721	3.030	2-M2c	2.500	1.700	1.703	2.470	7.017	0.000
172.00	29.68	1.71	3.135	3.587	7-M2c	2.500	1.850	1.857	2.470	7.590	0.000
230.00	38.90	2.98	4.151	4.857	7-M2c	2.500	2.090	2.104	2.470	8.822	0.000
275.00	41.16	3.32	4.446	5.204	7-M2c	2.500	2.137	2.151	2.470	9.161	0.000
320.00	41.73	3.41	4.524	5.292	7-M2c	2.500	2.149	2.166	2.470	9.237	0.000
365.00	42.20	3.49	4.588	5.365	7-M2c	2.500	2.159	2.175	2.470	9.307	0.000
410.00	42.61	3.55	4.645	5.430	7-M2c	2.500	2.168	2.184	2.470	9.369	0.000
455.00	42.98	3.61	4.698	5.489	7-M2c	2.500	2.176	2.191	2.470	9.426	0.000
500.00	43.33	3.66	4.746	5.543	7-M2c	2.500	2.183	2.198	2.470	9.477	0.000

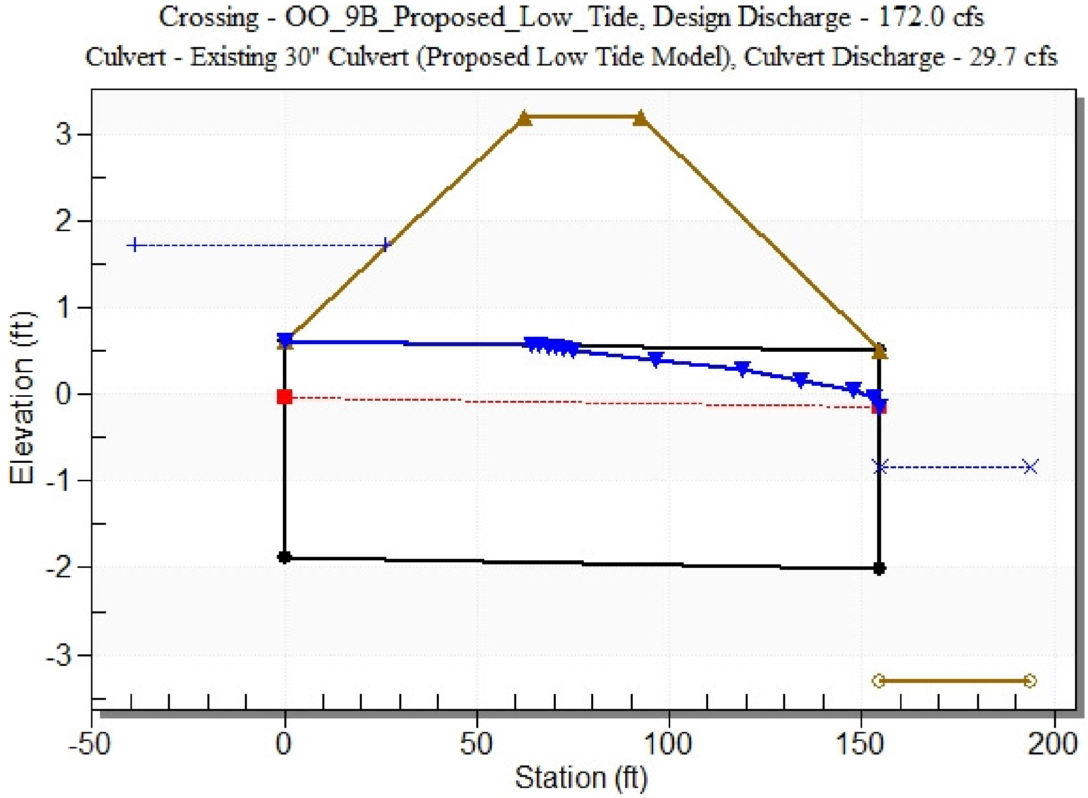
****

Inlet Elevation (invert): -1.88 ft, Outlet Elevation (invert): -2.00 ft

Culvert Length: 155.00 ft, Culvert Slope: 0.0008

****

Water Surface Profile Plot for Culvert: Existing 30" Culvert (Proposed Low Tide Model)



Site Data - Existing 30" Culvert (Proposed Low Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.88 ft

Outlet Station: 155.00 ft

Outlet Elevation: -2.00 ft

Number of Barrels: 1

Culvert Data Summary - Existing 30" Culvert (Proposed Low Tide Model)

Barrel Shape: Circular

Barrel Diameter: 2.50 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: NONE

Table HY8-8 - Downstream Channel Rating Curve (Crossing: OO_9B_Proposed_Low_Tide)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
50.00	-0.83	2.47
95.00	-0.83	2.47
140.00	-0.83	2.47
172.00	-0.83	2.47
230.00	-0.83	2.47
275.00	-0.83	2.47
320.00	-0.83	2.47
365.00	-0.83	2.47
410.00	-0.83	2.47
455.00	-0.83	2.47
500.00	-0.83	2.47

Tailwater Channel Data - OO_9B_Proposed_Low_Tide

Tailwater Channel Option: Enter Rating Curve

Roadway Data for Crossing: OO_9B_Proposed_Low_Tide

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 260.00 ft

Crest Elevation: 3.20 ft

Roadway Surface: Paved

Roadway Top Width: 30.00 ft

Table HY8-9 - Summary of Culvert Flows at Crossing: OO_9B_Proposed_High_Tide

Headwater Elevation (ft)	Total Discharge (cfs)	OO_09_Proposed (Proposed High Tide Model) Discharge (cfs)	Existing 30" Culvert (Proposed High Tide Model) Discharge (cfs)	Roadway Discharge (cfs)	Iterations
1.98	50.00	41.00	9.17	0.00	10
2.36	95.00	77.65	17.38	0.00	6
2.97	140.00	114.28	25.58	0.00	12
3.21	155.00	125.53	28.10	0.74	29
3.39	230.00	133.62	29.92	65.82	6
3.47	275.00	136.76	30.61	107.41	5
3.53	320.00	139.41	31.21	149.00	4
3.59	365.00	141.81	31.74	191.28	4
3.65	410.00	143.97	32.23	233.16	3
3.70	455.00	146.03	32.69	275.80	3
3.75	500.00	147.97	33.12	318.64	3
3.20	153.07	125.07	27.99	0.00	Overtopping

Table HY8-10 - Culvert Summary Table: OO_09_Proposed (Proposed High Tide Model)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	41.00	1.98	1.630	3.877	4-FFf	1.049	1.174	3.000	5.130	1.933	0.000
95.00	77.65	2.36	2.382	4.258	4-FFf	1.506	1.636	3.000	5.130	3.662	0.000
140.00	114.28	2.97	3.034	4.874	4-FFf	1.925	2.003	3.000	5.130	5.389	0.000
155.00	125.53	3.21	3.251	5.110	4-FFf	2.057	2.106	3.000	5.130	5.919	0.000
230.00	133.62	3.39	3.417	5.293	4-FFf	2.162	2.168	3.000	5.130	6.301	0.000
275.00	136.76	3.47	3.484	5.368	4-FFf	2.205	2.192	3.000	5.130	6.449	0.000
320.00	139.41	3.53	3.542	5.432	4-FFf	2.242	2.212	3.000	5.130	6.574	0.000
365.00	141.81	3.59	3.595	5.491	4-FFf	2.275	2.230	3.000	5.130	6.687	0.000
410.00	143.97	3.65	3.643	5.545	4-FFf	2.305	2.247	3.000	5.130	6.789	0.000
455.00	146.03	3.70	3.690	5.597	4-FFf	2.334	2.263	3.000	5.130	6.886	0.000
500.00	147.97	3.75	3.735	5.647	4-FFf	2.361	2.277	3.000	5.130	6.978	0.000

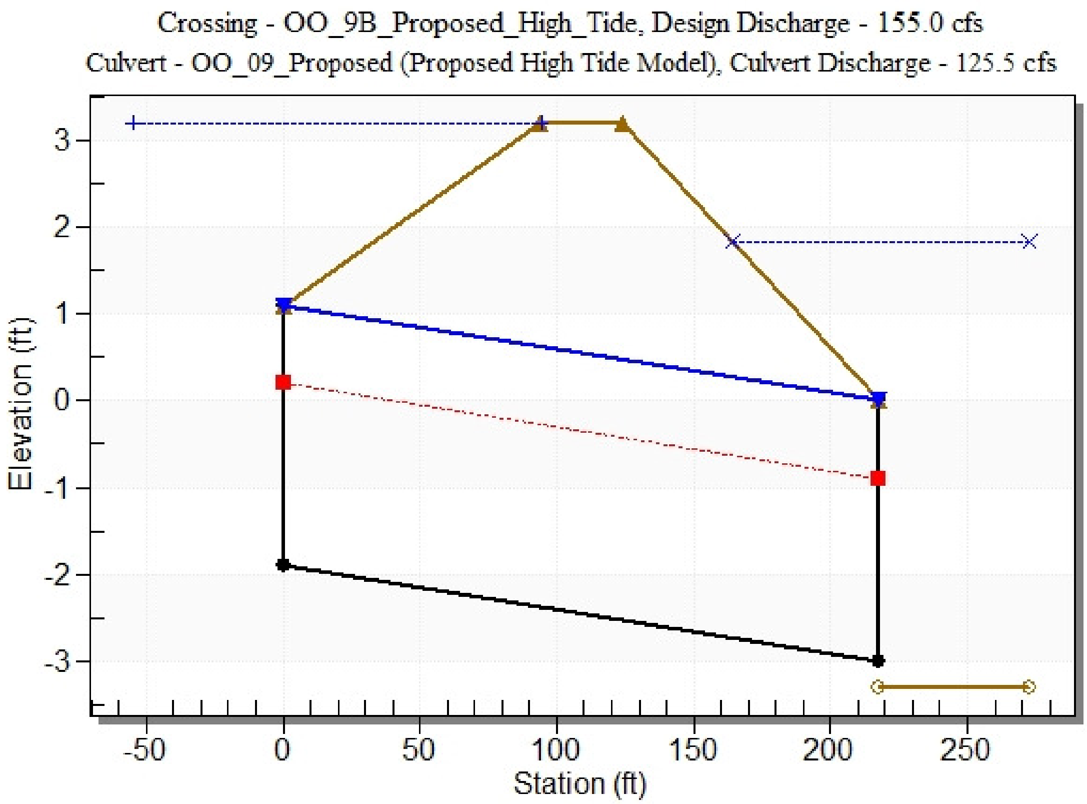
****

Inlet Elevation (invert): -1.90 ft,   Outlet Elevation (invert): -3.00 ft

Culvert Length: 218.00 ft,   Culvert Slope: 0.0050

****

Water Surface Profile Plot for Culvert: OO_09_Proposed (Proposed High Tide Model)



Site Data - OO_09_Proposed (Proposed High Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.90 ft

Outlet Station: 218.00 ft

Outlet Elevation: -3.00 ft

Number of Barrels: 3

Culvert Data Summary - OO_09_Proposed (Proposed High Tide Model)

Barrel Shape: Circular

Barrel Diameter: 3.00 ft

Barrel Material: Smooth HDPE

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Beveled Edge (1:1)

Inlet Depression: NONE

Table HY8-11 - Culvert Summary Table: Existing 30" Culvert (Proposed High Tide Model)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	9.17	1.98	1.433	3.857	4-FFf	1.604	1.009	2.500	5.130	1.869	0.000
95.00	17.38	2.36	2.125	4.238	4-FFf	2.500	1.406	2.500	5.130	3.541	0.000
140.00	25.58	2.97	2.770	4.853	4-FFf	2.500	1.721	2.500	5.130	5.212	0.000
155.00	28.10	3.21	2.989	5.089	4-FFf	2.500	1.802	2.500	5.130	5.724	0.000
230.00	29.92	3.39	3.158	5.274	4-FFf	2.500	1.857	2.500	5.130	6.094	0.000
275.00	30.61	3.47	3.226	5.348	4-FFf	2.500	1.878	2.500	5.130	6.237	0.000
320.00	31.21	3.53	3.284	5.412	4-FFf	2.500	1.896	2.500	5.130	6.358	0.000
365.00	31.74	3.59	3.338	5.470	4-FFf	2.500	1.912	2.500	5.130	6.466	0.000
410.00	32.23	3.65	3.387	5.524	4-FFf	2.500	1.927	2.500	5.130	6.565	0.000
455.00	32.69	3.70	3.435	5.577	4-FFf	2.500	1.941	2.500	5.130	6.659	0.000
500.00	33.12	3.75	3.481	5.627	4-FFf	2.500	1.954	2.500	5.130	6.748	0.000

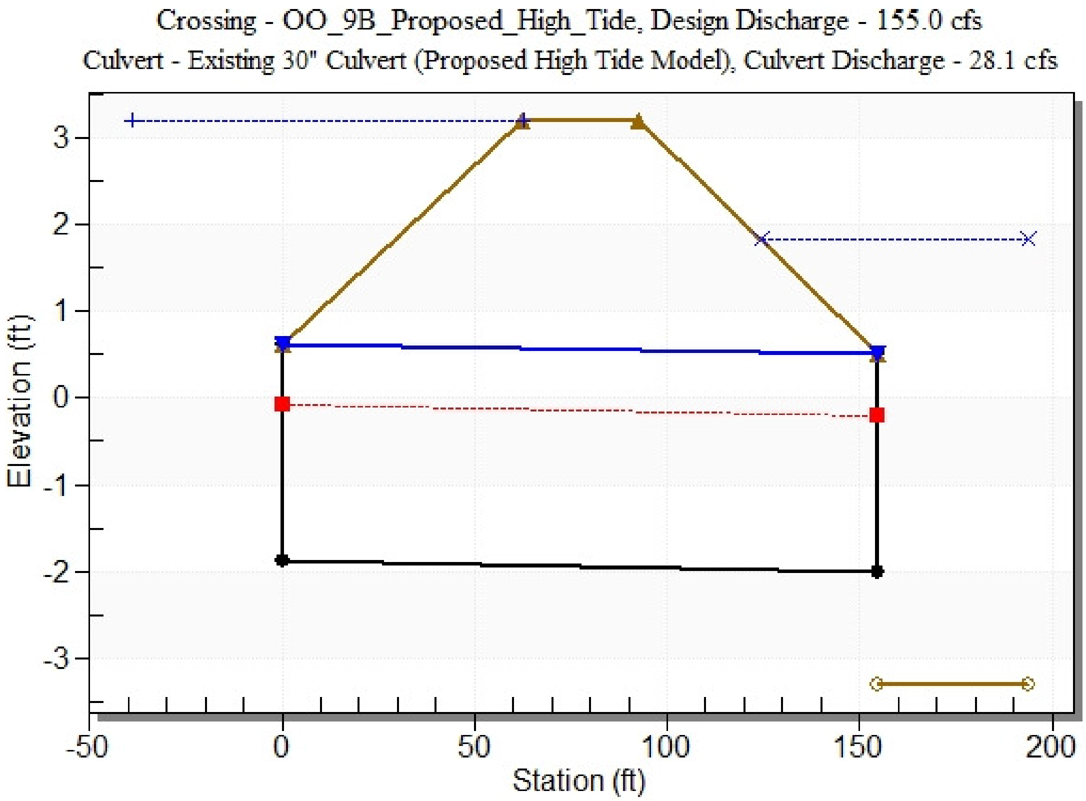
****

Inlet Elevation (invert): -1.88 ft, Outlet Elevation (invert): -2.00 ft

Culvert Length: 155.00 ft, Culvert Slope: 0.0008

****

Water Surface Profile Plot for Culvert: Existing 30" Culvert (Proposed High Tide Model)



Site Data - Existing 30" Culvert (Proposed High Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.88 ft

Outlet Station: 155.00 ft

Outlet Elevation: -2.00 ft

Number of Barrels: 1

Culvert Data Summary - Existing 30" Culvert (Proposed High Tide Model)

Barrel Shape: Circular

Barrel Diameter: 2.50 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: NONE

Table HY8-12 - Downstream Channel Rating Curve (Crossing: OO_9B_Proposed_High_Tide)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
50.00	1.83	5.13
95.00	1.83	5.13
140.00	1.83	5.13
155.00	1.83	5.13
230.00	1.83	5.13
275.00	1.83	5.13
320.00	1.83	5.13
365.00	1.83	5.13
410.00	1.83	5.13
455.00	1.83	5.13
500.00	1.83	5.13

Tailwater Channel Data - OO_9B_Proposed_High_Tide

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 1.83 ft

Roadway Data for Crossing: OO_9B_Proposed_High_Tide

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 260.00 ft

Crest Elevation: 3.20 ft

Roadway Surface: Paved

Roadway Top Width: 30.00 ft

Table HY8-13 - Summary of Culvert Flows at Crossing: OO_9B_Existing_Average_Tide

Headwater Elevation (ft)	Total Discharge (cfs)	OO_09_Existing (Existing Average Tide Model) Discharge (cfs)	Roadway Discharge (cfs)	Iterations
3.65	50.00	42.45	7.09	35
3.80	95.00	43.45	50.85	11
3.85	130.00	43.82	85.54	7
3.92	185.00	44.23	139.48	5
3.96	230.00	44.49	184.60	4
3.99	275.00	44.70	229.91	4
4.02	320.00	44.89	274.02	3
4.05	365.00	45.07	319.28	3
4.08	410.00	45.24	364.44	3
4.10	455.00	45.40	409.48	3
4.12	500.00	45.54	453.25	2
3.51	41.50	41.50	0.00	Overtopping

Table HY8-14 - Culvert Summary Table: OO_09_Existing (Existing Average Tide Model)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	42.45	3.65	4.623	5.529	4-FFf	2.500	2.164	2.500	3.800	8.649	0.000
95.00	43.45	3.80	4.764	5.678	4-FFf	2.500	2.185	2.500	3.800	8.851	0.000
130.00	43.82	3.85	4.817	5.735	4-FFf	2.500	2.193	2.500	3.800	8.926	0.000
185.00	44.23	3.92	4.877	5.798	4-FFf	2.500	2.202	2.500	3.800	9.011	0.000
230.00	44.49	3.96	4.914	5.838	4-FFf	2.500	2.207	2.500	3.800	9.062	0.000
275.00	44.70	3.99	4.947	5.872	4-FFf	2.500	2.212	2.500	3.800	9.107	0.000
320.00	44.89	4.02	4.975	5.902	4-FFf	2.500	2.216	2.500	3.800	9.146	0.000
365.00	45.07	4.05	5.002	5.930	4-FFf	2.500	2.220	2.500	3.800	9.182	0.000
410.00	45.24	4.08	5.026	5.956	4-FFf	2.500	2.223	2.500	3.800	9.216	0.000
455.00	45.40	4.10	5.050	5.981	4-FFf	2.500	2.227	2.500	3.800	9.248	0.000
500.00	45.54	4.12	5.071	6.004	4-FFf	2.500	2.230	2.500	3.800	9.277	0.000

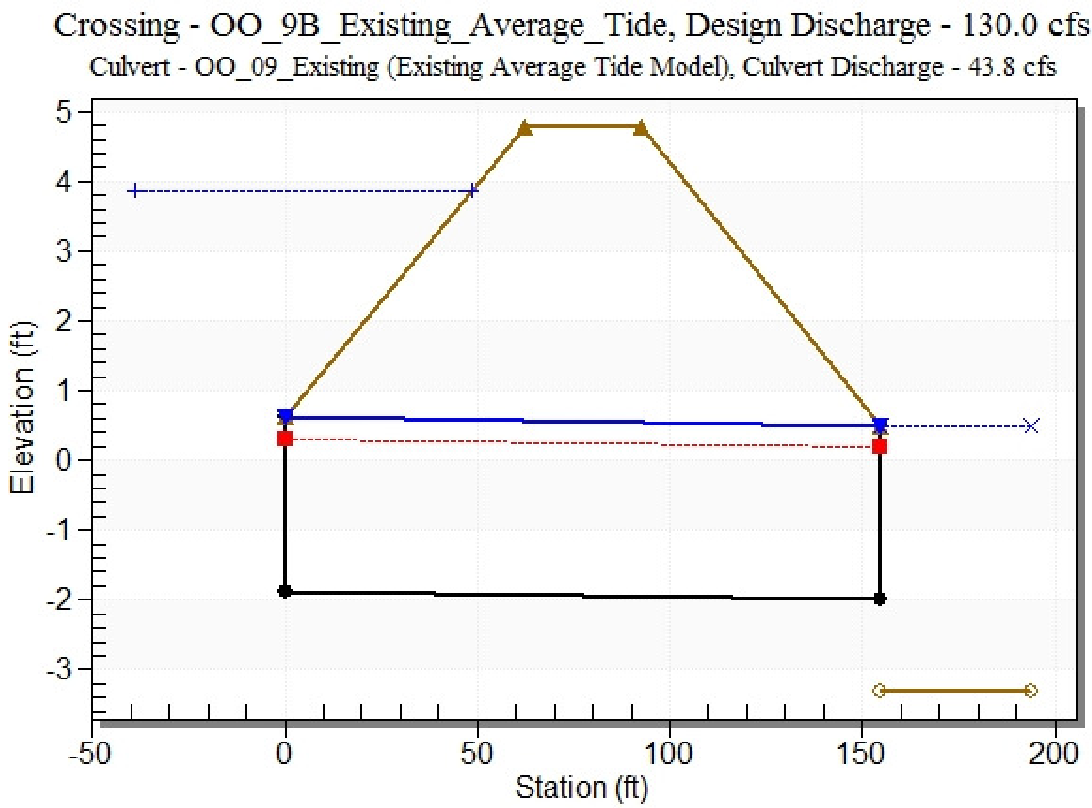
****

Inlet Elevation (invert): -1.88 ft, Outlet Elevation (invert): -2.00 ft

Culvert Length: 155.00 ft, Culvert Slope: 0.0008

****

Water Surface Profile Plot for Culvert: OO_09_Existing (Existing Average Tide Model)



Site Data - OO_09_Existing (Existing Average Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.88 ft

Outlet Station: 155.00 ft

Outlet Elevation: -2.00 ft

Number of Barrels: 1

Culvert Data Summary - OO_09_Existing (Existing Average Tide Model)

Barrel Shape: Circular

Barrel Diameter: 2.50 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: NONE

Table HY8-15 - Downstream Channel Rating Curve (Crossing: OO_9B_Existing_Average_Tide)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
50.00	0.50	3.80
95.00	0.50	3.80
130.00	0.50	3.80
185.00	0.50	3.80
230.00	0.50	3.80
275.00	0.50	3.80
320.00	0.50	3.80
365.00	0.50	3.80
410.00	0.50	3.80
455.00	0.50	3.80
500.00	0.50	3.80

Tailwater Channel Data - OO_9B_Existing_Average_Tide

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 0.50 ft

Roadway Data for Crossing: OO_9B_Existing_Average_Tide

Roadway Profile Shape: Irregular Roadway Shape (coordinates)

Irregular Roadway Cross-Section:

Coord No.	Station (ft)	Elevation (ft)
0	0.00	4.79
1	87.00	4.14
2	180.00	3.87
3	266.00	3.92
4	358.00	3.90
5	434.00	3.66
6	524.00	4.16
7	677.00	3.51
8	833.00	3.73
9	951.00	3.83

Roadway Surface: Paved

Roadway Top Width: 30.00 ft

Table HY8-16 - Summary of Culvert Flows at Crossing: OO_9B_Existing_Low_Tide

Headwater Elevation (ft)	Total Discharge (cfs)	OO_09_Existing (Existing Low Tide Model) Discharge (cfs)	Roadway Discharge (cfs)	Iterations
3.64	50.00	43.20	6.35	61
3.80	95.00	44.16	50.07	11
3.85	130.00	44.51	84.83	7
3.92	185.00	44.90	138.77	5
3.96	230.00	45.15	183.92	4
3.99	275.00	45.36	229.25	4
4.02	320.00	45.54	273.37	3
4.05	365.00	45.71	318.63	3
4.08	410.00	45.87	363.81	3
4.10	455.00	46.02	408.85	3
4.12	500.00	46.16	452.61	2
3.51	42.36	42.36	0.00	Overtopping

Table HY8-17 - Culvert Summary Table: OO_09_Existing (Existing Low Tide Model)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	43.20	3.64	4.729	5.523	7-M2c	2.500	2.180	2.195	2.470	9.459	0.000
95.00	44.16	3.80	4.866	5.677	7-M2c	2.500	2.200	2.209	2.470	9.620	0.000
130.00	44.51	3.85	4.918	5.734	7-M2c	2.500	2.208	2.215	2.470	9.677	0.000
185.00	44.90	3.92	4.976	5.798	7-M2c	2.500	2.216	2.221	2.470	9.742	0.000
230.00	45.15	3.96	5.012	5.837	7-M2c	2.500	2.221	2.225	2.470	9.782	0.000
275.00	45.36	3.99	5.044	5.871	7-M2c	2.500	2.226	2.228	2.470	9.817	0.000
320.00	45.54	4.02	5.071	5.901	7-M2c	2.500	2.230	2.236	2.470	9.831	0.000
365.00	45.71	4.05	5.098	5.929	7-M2c	2.500	2.233	2.239	2.470	9.859	0.000
410.00	45.87	4.08	5.122	5.956	7-M2c	2.500	2.237	2.242	2.470	9.884	0.000
455.00	46.02	4.10	5.145	5.980	7-M2c	2.500	2.240	2.245	2.470	9.908	0.000
500.00	46.16	4.12	5.166	6.003	7-M2c	2.500	2.243	2.247	2.470	9.930	0.000

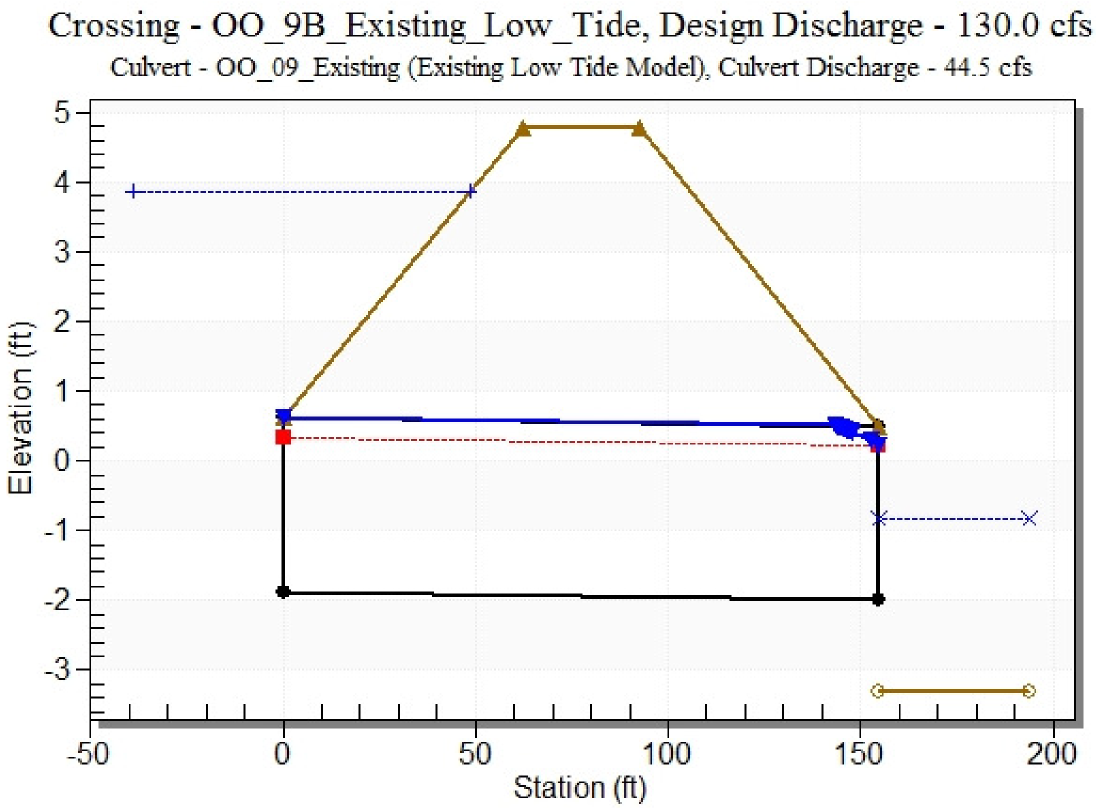
****

Inlet Elevation (invert): -1.88 ft, Outlet Elevation (invert): -2.00 ft

Culvert Length: 155.00 ft, Culvert Slope: 0.0008

****

Water Surface Profile Plot for Culvert: OO_09_Existing (Existing Low Tide Model)



Site Data - OO_09_Existing (Existing Low Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.88 ft

Outlet Station: 155.00 ft

Outlet Elevation: -2.00 ft

Number of Barrels: 1

Culvert Data Summary - OO_09_Existing (Existing Low Tide Model)

Barrel Shape: Circular

Barrel Diameter: 2.50 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: NONE

Table HY8-18 - Downstream Channel Rating Curve (Crossing: OO_9B_Existing_Low_Tide)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
50.00	-0.83	2.47
95.00	-0.83	2.47
130.00	-0.83	2.47
185.00	-0.83	2.47
230.00	-0.83	2.47
275.00	-0.83	2.47
320.00	-0.83	2.47
365.00	-0.83	2.47
410.00	-0.83	2.47
455.00	-0.83	2.47
500.00	-0.83	2.47

Tailwater Channel Data - OO_9B_Existing_Low_Tide

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: -0.83 ft

Roadway Data for Crossing: OO_9B_Existing_Low_Tide

Roadway Profile Shape: Irregular Roadway Shape (coordinates)

Irregular Roadway Cross-Section:

Coord No.	Station (ft)	Elevation (ft)
0	0.00	4.79
1	87.00	4.14
2	180.00	3.87
3	266.00	3.92
4	358.00	3.90
5	434.00	3.66
6	524.00	4.16
7	677.00	3.51
8	833.00	3.73
9	951.00	3.83

Roadway Surface: Paved

Roadway Top Width: 30.00 ft

Table HY8-19 - Summary of Culvert Flows at Crossing: OO_9B_Existing_High_Tide

Headwater Elevation (ft)	Total Discharge (cfs)	OO_09_Existing (Existing High Tide Model) Discharge (cfs)	Roadway Discharge (cfs)	Iterations
3.71	50.00	32.77	16.93	12
3.82	95.00	33.71	60.42	9
3.87	130.00	34.14	94.83	6
3.93	185.00	34.65	149.47	5
3.97	230.00	34.96	194.31	4
4.00	275.00	35.22	238.03	3
4.03	320.00	35.46	283.41	3
4.06	365.00	35.69	328.74	3
4.08	410.00	35.90	373.84	3
4.11	455.00	36.09	418.81	3
4.13	500.00	36.26	462.77	2
3.51	31.01	31.01	0.00	Overtopping

Table HY8-20 - Culvert Summary Table: OO_09_Existing (Existing High Tide Model)

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
50.00	32.77	3.71	3.443	5.586	4-FFf	2.500	1.943	2.500	5.130	6.675	0.000
95.00	33.71	3.82	3.544	5.696	4-FFf	2.500	1.971	2.500	5.130	6.868	0.000
130.00	34.14	3.87	3.591	5.747	4-FFf	2.500	1.985	2.500	5.130	6.956	0.000
185.00	34.65	3.93	3.646	5.808	4-FFf	2.500	2.000	2.500	5.130	7.059	0.000
230.00	34.96	3.97	3.681	5.845	4-FFf	2.500	2.006	2.500	5.130	7.122	0.000
275.00	35.22	4.00	3.710	5.877	4-FFf	2.500	2.012	2.500	5.130	7.175	0.000
320.00	35.46	4.03	3.738	5.908	4-FFf	2.500	2.017	2.500	5.130	7.225	0.000
365.00	35.69	4.06	3.764	5.935	4-FFf	2.500	2.022	2.500	5.130	7.270	0.000
410.00	35.90	4.08	3.787	5.961	4-FFf	2.500	2.026	2.500	5.130	7.313	0.000
455.00	36.09	4.11	3.810	5.986	4-FFf	2.500	2.030	2.500	5.130	7.352	0.000
500.00	36.26	4.13	3.830	6.008	4-FFf	2.500	2.034	2.500	5.130	7.387	0.000

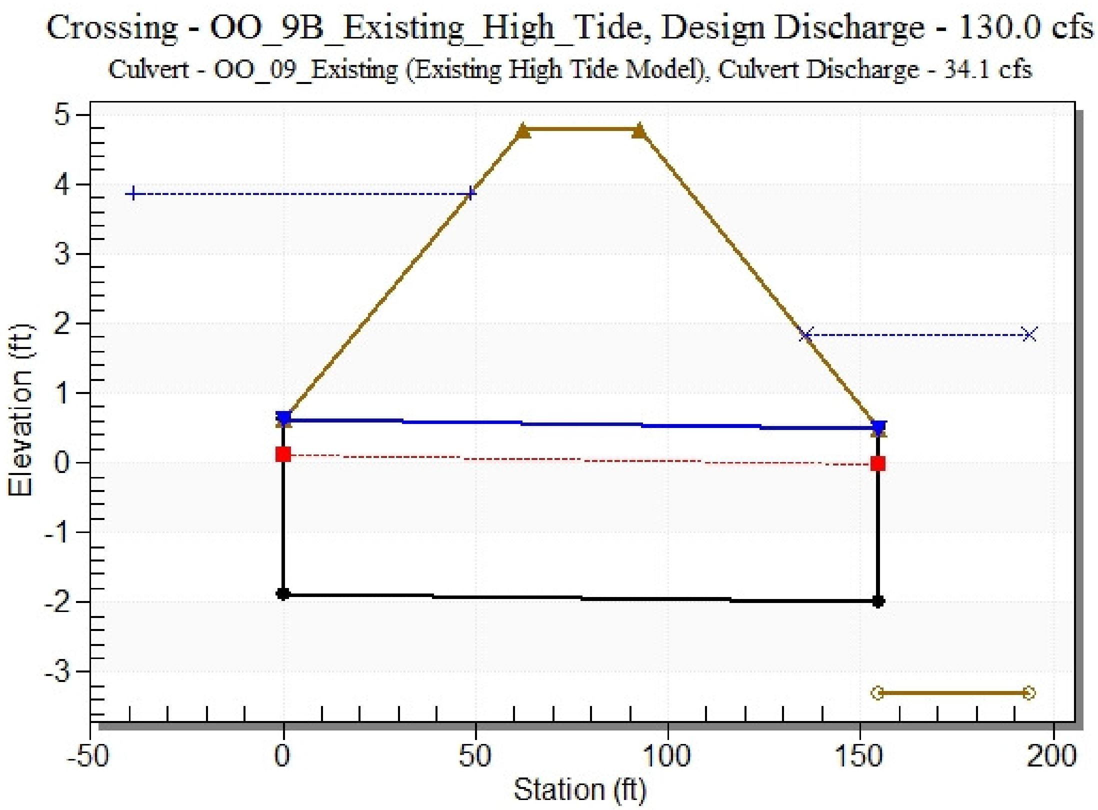
****

Inlet Elevation (invert): -1.88 ft, Outlet Elevation (invert): -2.00 ft

Culvert Length: 155.00 ft, Culvert Slope: 0.0008

****

Water Surface Profile Plot for Culvert: OO_09_Existing (Existing High Tide Model)



Site Data - OO_09_Existing (Existing High Tide Model)

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: -1.88 ft

Outlet Station: 155.00 ft

Outlet Elevation: -2.00 ft

Number of Barrels: 1

Culvert Data Summary - OO_09_Existing (Existing High Tide Model)

Barrel Shape: Circular

Barrel Diameter: 2.50 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge with Headwall

Inlet Depression: NONE

Table HY8-21 - Downstream Channel Rating Curve (Crossing: OO_9B_Existing_High_Tide)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
50.00	1.83	5.13
95.00	1.83	5.13
130.00	1.83	5.13
185.00	1.83	5.13
230.00	1.83	5.13
275.00	1.83	5.13
320.00	1.83	5.13
365.00	1.83	5.13
410.00	1.83	5.13
455.00	1.83	5.13
500.00	1.83	5.13

Tailwater Channel Data - OO_9B_Existing_High_Tide

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 1.83 ft

Roadway Data for Crossing: OO_9B_Existing_High_Tide

Roadway Profile Shape: Irregular Roadway Shape (coordinates)

Irregular Roadway Cross-Section:

Coord No.	Station (ft)	Elevation (ft)
0	0.00	4.79
1	87.00	4.14
2	180.00	3.87
3	266.00	3.92
4	358.00	3.90
5	434.00	3.66
6	524.00	4.16
7	677.00	3.51
8	833.00	3.73
9	951.00	3.83

Roadway Surface: Paved

Roadway Top Width: 30.00 ft

Concept Design OO_12 / OO_13

1 EXISTING SITE DESCRIPTION

The questionnaire responses indicate that ponding occurs on neighborhood streets in various areas of Oak Orchard several times a year. Of particular concern is Forest Drive at its intersection with Delaware Street. Water was observed ponding at this intersection during both field investigations in September 2014 and February 2015. This area is approximately 200 feet from the nearest inlet, and there are no conveyances in the vicinity for it to drain to. Figure 1 and the photographs show existing conditions.

Ponding at the intersection of Forest Drive and Delaware Street and along Delaware Street, where two catch basins piped to the existing open channel along Paul Street are proposed

Farther downstream, a series of open channels and small-diameter (size not determined) pipes drain portions of Delaware, Paul, and Charles Streets and Mercer Avenue. These conveyances join at an inlet just off the eastern shoulder of Mercer Avenue and proceeds by an approximately 15-inch pipe to an outlet through the bulkhead at Indian River Bay.

The roadside channels along Paul Street are generally well defined but require some minor maintenance, including clearing of phragmites. During high tide, these channels and the inlets on Paul Street and Mercer Avenue receive river water from the Indian River Bay. Both the channels and inlets had standing water during field investigations.

The outfall pipe was almost entirely covered by sand and debris during the February 2015 visit. Based on input from local residents, it appears the pipe only drains when the sand is cleared away by those residents.

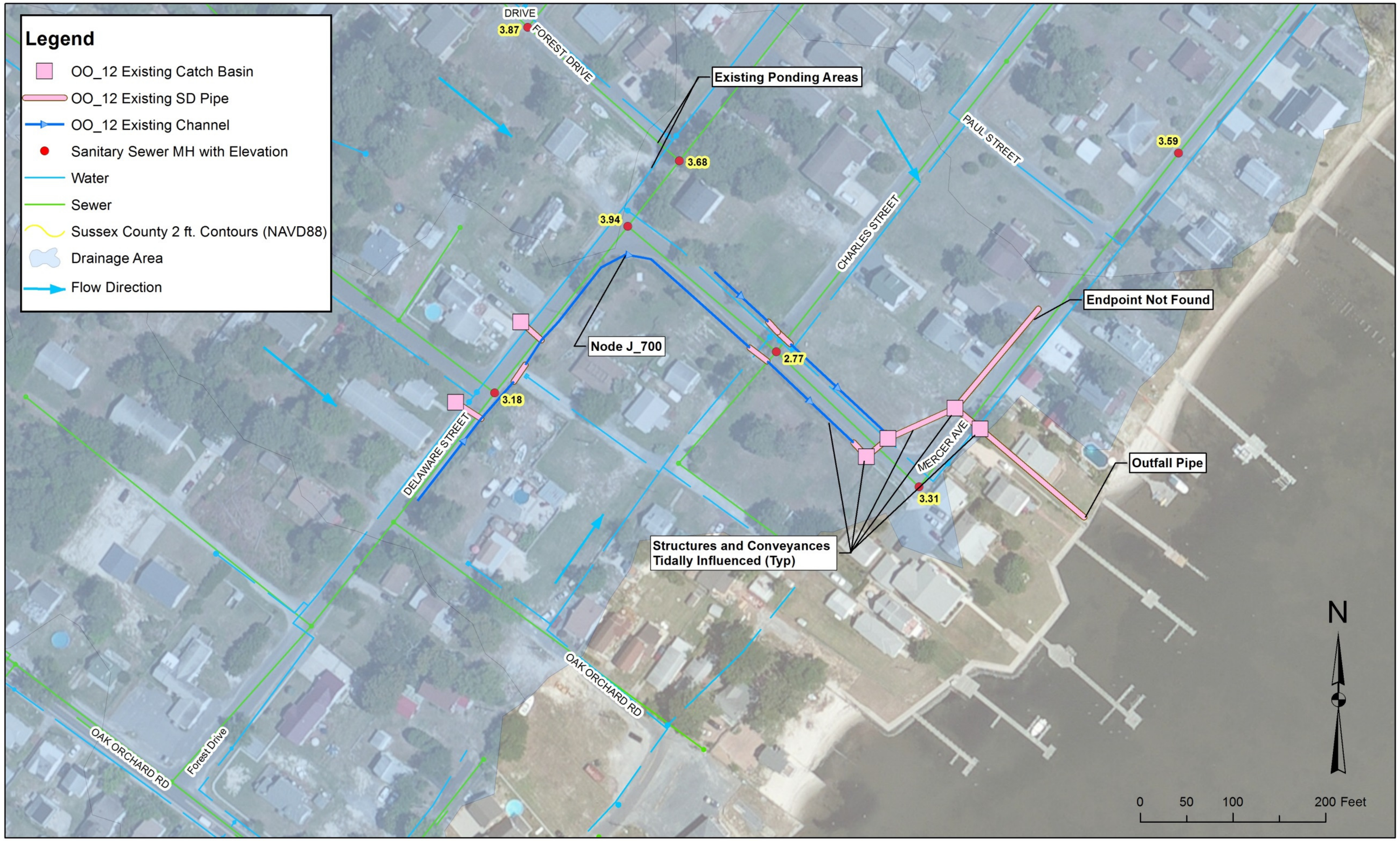


Figure 1: OO_12 / OO_13 Existing Site Conditions

2 PROPOSED IMPROVEMENT

Proposed improvements are shown in Figure 2. Two new catch basins will be installed, one at the western corner of Forest Drive and Delaware Street and the other 50 to 100 feet southwest of this location. These will be connected by a pipe that will discharge under the intersection of Delaware and Paul Streets to the existing channel on the west side of Paul Street. The drainage area and runoff rates to this intersection will need to be determined as part of final design; however, we anticipate that the increased flows from the proposed discharge pipe will place the existing downstream pipes over capacity. Therefore, the proposed design should include a program to clean the existing channels along Mercer Avenue and Delaware Street and replacing the existing storm drain system with 18-inch diameter pipes at a 0.5 percent slope. Invasive phragmites in the channels will also be removed and riprap placed at pipe inlets and outlets.

Extending the outfall pipe farther into the river would reduce the need to periodically clear sand

To eliminate the need for continual clearing of sediment from the outfall pipe, the pipe into the bay will be extended. Some minor restoration will be needed at the bulkhead as part of this work. A sediment transport study will be required to verify the extent of the deposition zone and length of pipe required. A pier will need to be installed over the proposed pipe to provide maintenance access. Because of tidal influences on the system, a backflow prevention device (e.g., Tideflex CheckMate valve) is proposed.

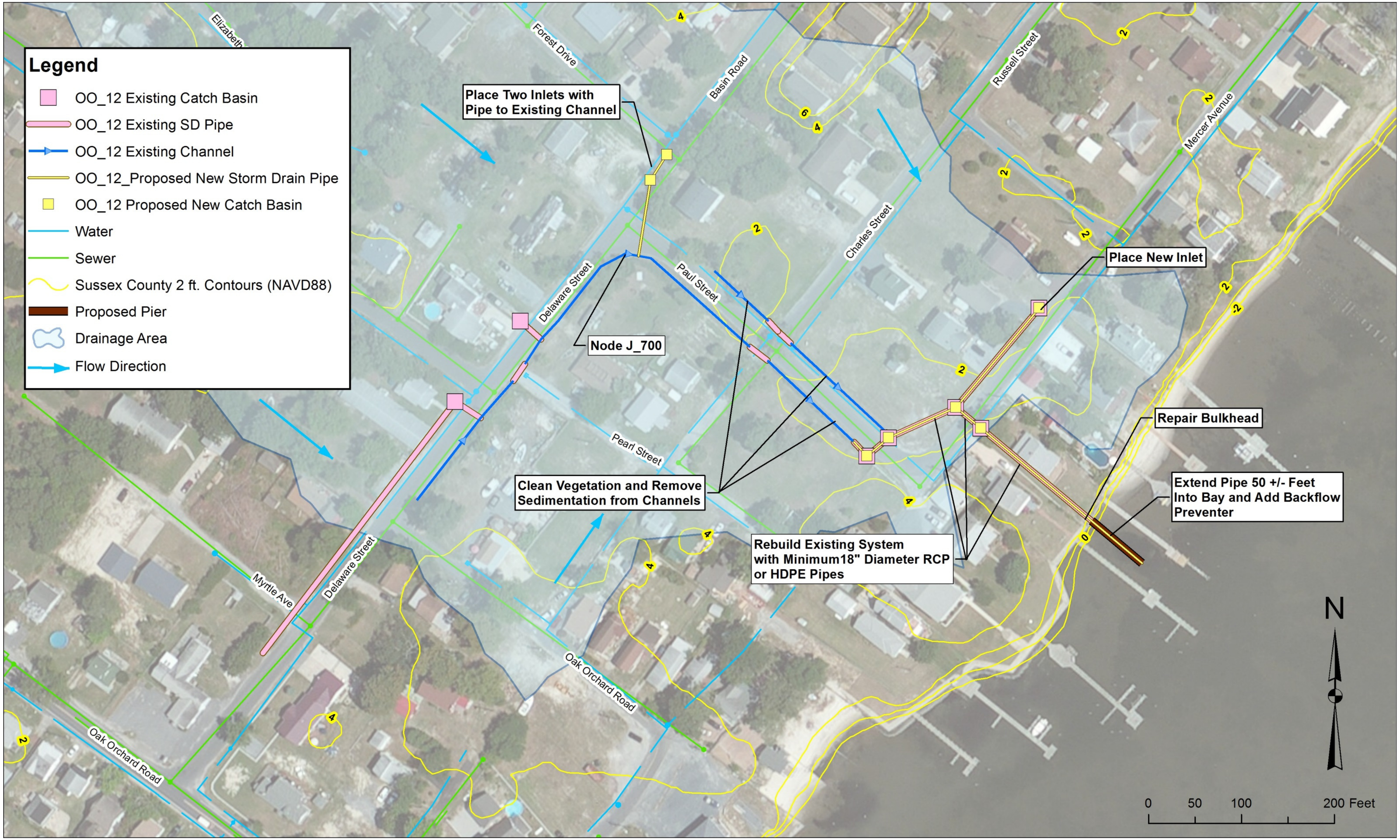


Figure 2: OO_12 / OO_13 Proposed Site Design

3 HYDRAULIC AND HYDROLOGIC CALCULATIONS

3.1 HEC-HMS Hydrologic Analysis

A hydrologic analysis for Oak Orchard was performed using HEC-HMS. The methodology and results of this study are discussed in Appendix F. The peak flows to the intersection of Forest Drive and Delaware Street (node J_700) and at the conveyance system outfall (node Outfall 7) are shown in Table 1:

Table 1 Peak Flow Rates

Name	Location Description	Drainage Area, mi^2	Storm Event Flows (cubic feet per second)				
			2-year	10-year	25-year	50-year	100-year
J_700	Downstream of Forest Drive	0.05	4	10	16	22	28
Outfall 7	Upstream of the Mercer Avenue Culvert	0.07	6	13	21	28	36

3.2 Tidal Tailwater Boundary Conditions

As evidenced by the presence of bay water in the existing conveyances, the hydraulics of this site are influenced by the Indian River Bay. The U.S. Geological Survey (USGS) Indian River stream gage at Rosedale Beach (Gage 01484540) is located less than a mile from the Oak Orchard community and was used to estimate average low tide, high tide, and overall average water surface elevations for the Indian River Bay (USGS, 2012). The gage data were provided in National Geodetic Vertical Datum of 1929 (NGVD29) and corrected to the North American Vertical Datum 1988 (NAVD88) by adding -0.78 foot to the NGVD29 elevation. According to daily data from 2006 to 2015, the average low tide elevation is -0.83 foot, the average high tide elevation is 1.83 feet, and the overall average water surface elevation is 0.5 foot.

The average high tide and average elevations are consistent with field observations that drainage conveyances are negatively influenced by tailwater effects. The proposed backflow prevention device is intended to stop tidal water from entering the system.

3.3 Existing Hydraulic Calculations

The Inteli Solve HydraFlow Express (2006) program, which is based on Manning's equation, was used to model the storm drain, culverts, and channels using the 10-year peak flow rates calculated with HEC-HMS as described above. To use HydraFlow Express, hydrologic input files had to be developed to match those in the URS HEC-HMS study for each catchment area using drainage area, time of concentration, and curve number. Peak flows were calculated at known points in the storm network.

3.4 Proposed Hydraulic Calculations

The proposed hydraulic conditions were calculated using the same method used for the existing hydraulic conditions. HydraFlow Express was used to size proposed storm drains, culverts, and channels to meet the permissible velocity and freeboard requirements specified in the Delaware Department of Transportation (DelDOT) Road Design Manual (2008). A 10-year design storm was used as the basis for our evaluations as specified in the manual for storm drain, culverts, and roadside channels along local roads.

An iterative approach had to be used to calculate the required pipe and channel sizes because of the limited topographic relief throughout the site and other site constraints. The channels should be constructed or redefined to the following trapezoidal dimensions:

- Bottom Width = 2 feet
- Side Slopes = 2H:1V
- Total Depth = 2 feet
- Slope = 0.5 %

The existing storm sewer should be replaced with 18-inch pipes at the maximum slope allowable by existing grades. These pipes could be either reinforced concrete or high-density polyethylene. The cost estimate in Section 7 is based on polyethylene. Material selection would be made by DelDOT.

4 IMPROVEMENTS AND BENEFITS

The proposed design would decrease the frequency and duration of ponding from localized runoff and bay tidal events. Cleaning the existing channels will allow them to flow at design capacity, thereby expediting the removal of runoff from the site. In addition, removal of invasive phragmites will allow native vegetation to develop. Replacing the existing pipe system will provide the additional capacity needed to carry runoff from the ponding areas not currently draining to this system. Installing backflow prevention at the outfall into the bay will prevent tidal waters from entering the storm drain system. Extending the outfall pipe should eliminate the need for residents to clear the outfall pipe.

5 FEASIBILITY ASSESSMENT

Soil and Groundwater: The soils at the proposed design and the drainage area are all Runclint loamy sand at 0 to 2 percent slopes (NRCS). Runclint loamy sand is classified as hydrologic soil group A, which are well drained soils primarily composed of sand. Sandy soils have no cohesion, so design velocities will need to be considered carefully during final design to avoid erosion. Groundwater data from the Delaware Geological Survey indicate that the water table is approximately 7 to 10 feet below the ground surface. Field exploration would be needed to confirm this. If actual elevations are higher, it could result in standing water in both the existing and proposed channels. In these situations, the channels would convey less water after precipitation events, but the design is still expected to expedite the removal of runoff from the site.

Construction Access: These locations are all easily accessible from public roads with the exception of the pipe between Mercer Avenue and the bay. Construction equipment may need to be parked along roadways. The new inlets at the intersection of Forest Drive and Delaware Street should be within the right-of-way.

Maintenance Considerations: Routine maintenance will be needed to keep the outfall storm pipe into the bay, and potentially the backflow preventer (depending on where it is placed) clear, although extending the outfall pipe should greatly reduce this need. Routine maintenance will also be required to sustain the design channel flow capacity. Maintenance would include periodically removing phragmites and sediment and cutting grass.

Utility Conflicts: There is a sanitary sewer system in the vicinity of these modifications. The proposed pipe from the new inlets near Forest Drive will cross this sewer, so elevations will need to be checked at final design. Other crossings of the storm sewer system already exist, and conflicts may develop if inverts are raised or lowered as part of the proposed design. Locations of any other below-ground utilities such as water or gas, or above-ground utilities such as electric lines, would need to be confirmed during detailed design.

Effectiveness: The proposed design would substantially reduce nuisance ponding from frequent storm events. Ponding from large coastal events would still be expected; however, the duration of ponding should be reduced. The effectiveness of the proposed design would be dependent on the effectiveness of the backflow prevention device in keeping bay water out of the system, as well as routine maintenance of the existing channel. Chemical treatment or other measures may be required to prevent phragmites from reestablishing in the channels.

Environmental Issues: Extending the existing pipe into the bay could disturb subaqueous areas. There are no potential tree impacts at this site. Invasive phragmites would be removed as part of this project. Remaining construction and maintenance would occur in areas already developed.

Easements: An easement will likely be needed for construction and maintenance of the pipe between Mercer Avenue and the bay if none already exists.

6 PLANS AND PERMITTING

Several construction documents and plans would need to be obtained to implement the proposed drainage design, including, but not limited to those described in Table 2.

Table 2: Required Plans and Permitting for Proposed Design OO_12 / OO_13

Plans/Permits	Permitting Agency	Notes and Potential Difficulties
Wetlands and Subaqueous Lands Permit	DNREC	Extension of the pipe into the river would involve working in subaqueous lands.
Traffic Control Plan	DelDOT	
Erosion and Sediment Control Plan	Sussex Conservation District	
Utility Construction Permit	DelDOT	Limited utility impacts are anticipated for this project.

7 COST ESTIMATE

Table 3 summarizes the costs associated with this concept design.

Table 3 Estimated Project Costs for OO_12 / OO_13

ITEM	QUANTITY	UNITS	UNIT COST		TOTAL
Remove and Dispose Existing Piping	400	LF	$20.00		$8,000
Regrade Existing Channels	650	LF	$10.00		$6,500
Asphalt Base	25	TON	$100.00		$2,500
Asphalt Surface	12	TON	$110.00		$1,320
Riprap	10	SY	$90.00		$900
Backflow Preventer Check Valve	1	EA	$3,300.00		$3,300
Cofferdams for River Construction	1	LS	$10,000.00		$10,000
Reconstruct Bulkhead	1	LS	$2,500.00		$2,500
Inlet	7	EA	$2,500.00		$17,500
Seeding and Mulching	8000	SY	$2.50		$20,000
Traffic Control Plan	5	DAY	$750.00		$3,750
High Density Polyethylene Pipe End Section	1	EA	$150.00		$150
High Density Polyethylene Pipe	550	LF	$45.00		$24,750
Maintenance Pier	1	EA	$15,000.00		$15,000
EA = each LF = linear foot SY = square yard LS = lump sum			Initial Project Costs		$116,170
			Contingency 10%		$11,617
		Erosion and Sediment Control		10%	$11,117
		Base Construction Costs			$139,404
		Mobilization		5%	$6,970
			Subtotal 1		$146,374
		Contingency		15%	$21,956
			Subtotal 2		$168,330
		Engineering			$50,000
				Total	$218,330[a,b]

[a] Sediment transport study costs are not included.
[b] Based on field location of existing utilities, relocation of the existing sewer line may be required, however these costs are not included.

8 REFERENCES

Delaware Department of Transportation (DelDOT), 2008. Road Design Manual Chapter Six: Drainage and Stormwater Management. Available at http://www.deldot.gov/information/pubs_forms/manuals/road_design/pdf/06_drainage_stormwater_mgmt.pdf

Intelisolve Hydraflow Express Version 1.08. Available at http://intelisolve.software.informer.com/

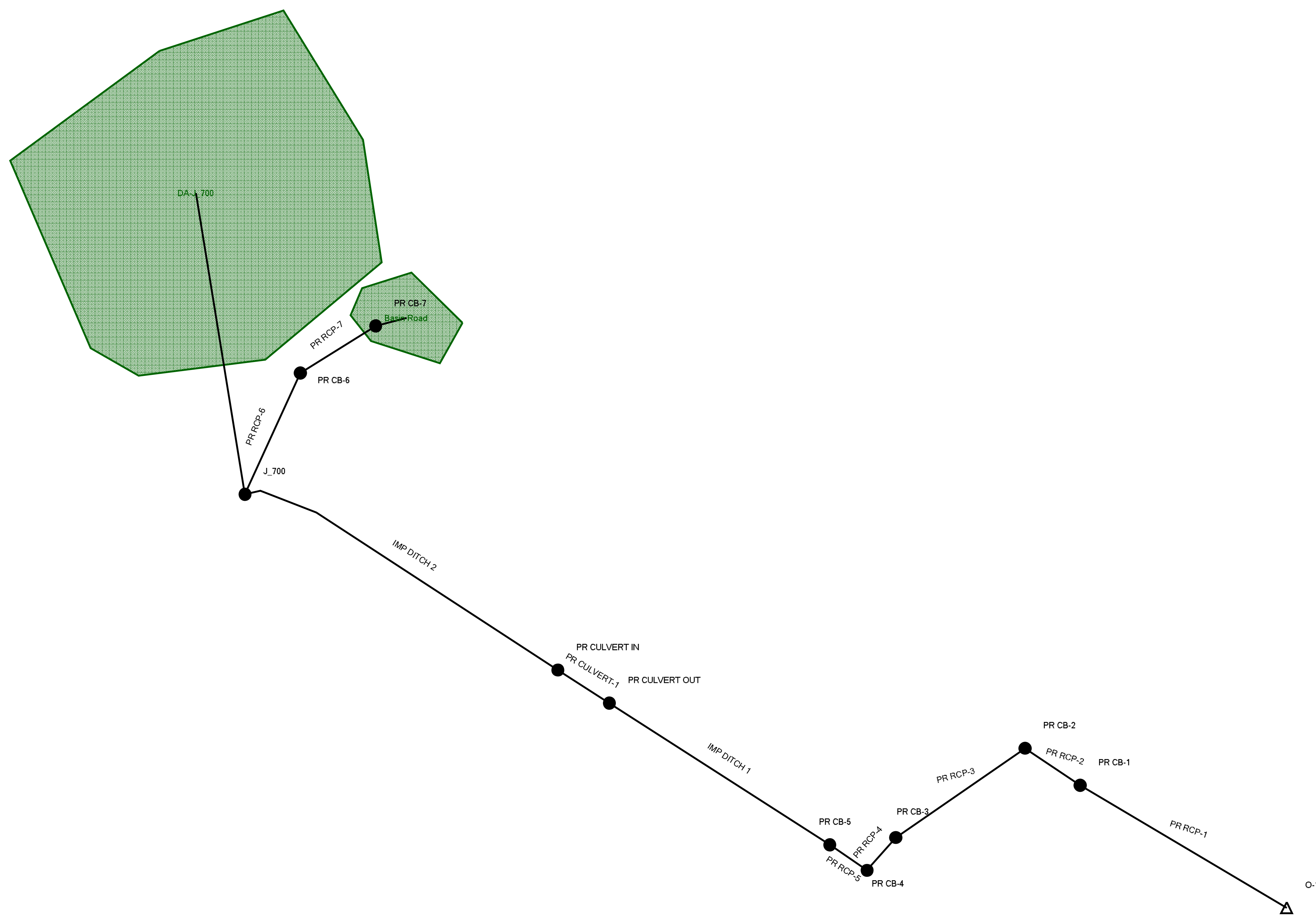
Water Table in the Inland Bays Watershed, Delaware. 2005. Delaware Geological Survey (DGS), accessed February 2015. Available at http://www.dgs.udel.edu/.

NRCS, 2009. U.S. General Soil Survey Geographic Database (SSURGO), accessed November 2014. Available at http://SoilDataMart.nrcs.usda.gov/.

U.S. Geological Survey (USGS), 2007. National Elevation Dataset (NED), accessed November 2014. Available at http://ned.usgs.gov/.

U.S. Geological Survey, 2012. National Water Information System data available on the World Wide Web (USGS Water Data for the Nation), accessed January 21, 2015. Available at http://waterdata.usgs.gov/nwis/

Scenario: Base



CONCEPT DESIGN #3: OO_12 and OO_13 (MERCER AVENUE AND FOREST DRIVE)

Project Summary	
Title	
Engineer	
Company	
Date	2/6/2015

Notes

Table of Contents

CONCEPT DESIGN #3: OO_12 and OO_13 (MERCER AVENUE AND FOREST DRIVE)

Subsection: Modified Rational Grand Summary

Modified Rational Method

Q = CiA * Units Conversion; Where conversion = 43560 / (12 * 3600)

Frequency (years)	Area (mile²)	Adjusted C Coefficient	Duration (hours)	Intensity (in/h)	Flow (Peak) (ft³/s)	Flow (Allowable) (ft³/s)	Volume (inflow) (ac-ft)	Volume (Storage) (ac-ft)
10	0.001	1.000	0.367	3.972	2.56	1.39	0.078	0.037
10	0.050	1.000	16.000	0.313	10.11	7.10	13.369	4.002

CONCEPT DESIGN #3: OO_12 and OO_13 (MERCER AVENUE AND FOREST DRIVE)

Subsection: Master Network Summary

Catchments Summary

Label	Scenario	Return Event (years)	Hydrograph Volume (ac-ft)	Time to Peak (hours)	Peak Flow (ft³/s)
Basin Road	Base	10	0.078	0.100	2.56
DA-J_700	Base	10	13.369	0.100	10.11

Node Summary

Label	Scenario	Return Event (years)	Hydrograph Volume (ac-ft)	Time to Peak (hours)	Peak Flow (ft³/s)
J_700	Base	10	13.447	0.300	12.67
O-1	Base	10	9.704	0.500	7.99
PR CB-1	Base	10	9.724	0.400	7.99
PR CB-2	Base	10	9.684	0.400	7.99
PR CB-3	Base	10	9.663	0.300	4.91
PR CB-4	Base	10	13.611	0.300	7.99
PR CB-5	Base	10	13.545	0.300	7.99
PR CB-6	Base	10	0.078	0.200	2.56
PR CB-7	Base	10	0.078	0.100	2.56
PR CULVERT IN	Base	10	13.447	0.400	12.57
PR CULVERT OUT	Base	10	13.545	0.200	7.99

CONCEPT DESIGN #3: OO_12 and OO_13 (MERCER AVENUE AND FOREST DRIVE)

Subsection: I-D-F Table

Return Event: 10 years

Label: Rainfall Intensity Sussex County

Storm Event: Rainfall Intensity Sussex County - 10 Year

I-D-F Curve

Time (hours)	Intensity (in/h)
0.083	6.760
0.167	5.400
0.250	4.560
0.500	3.300
1.000	2.150
2.000	1.350
3.000	0.990
6.000	0.610
12.000	0.360
24.000	0.220

CONCEPT DESIGN #3: OO_12 and OO_13 (MERCER AVENUE AND FOREST DRIVE)

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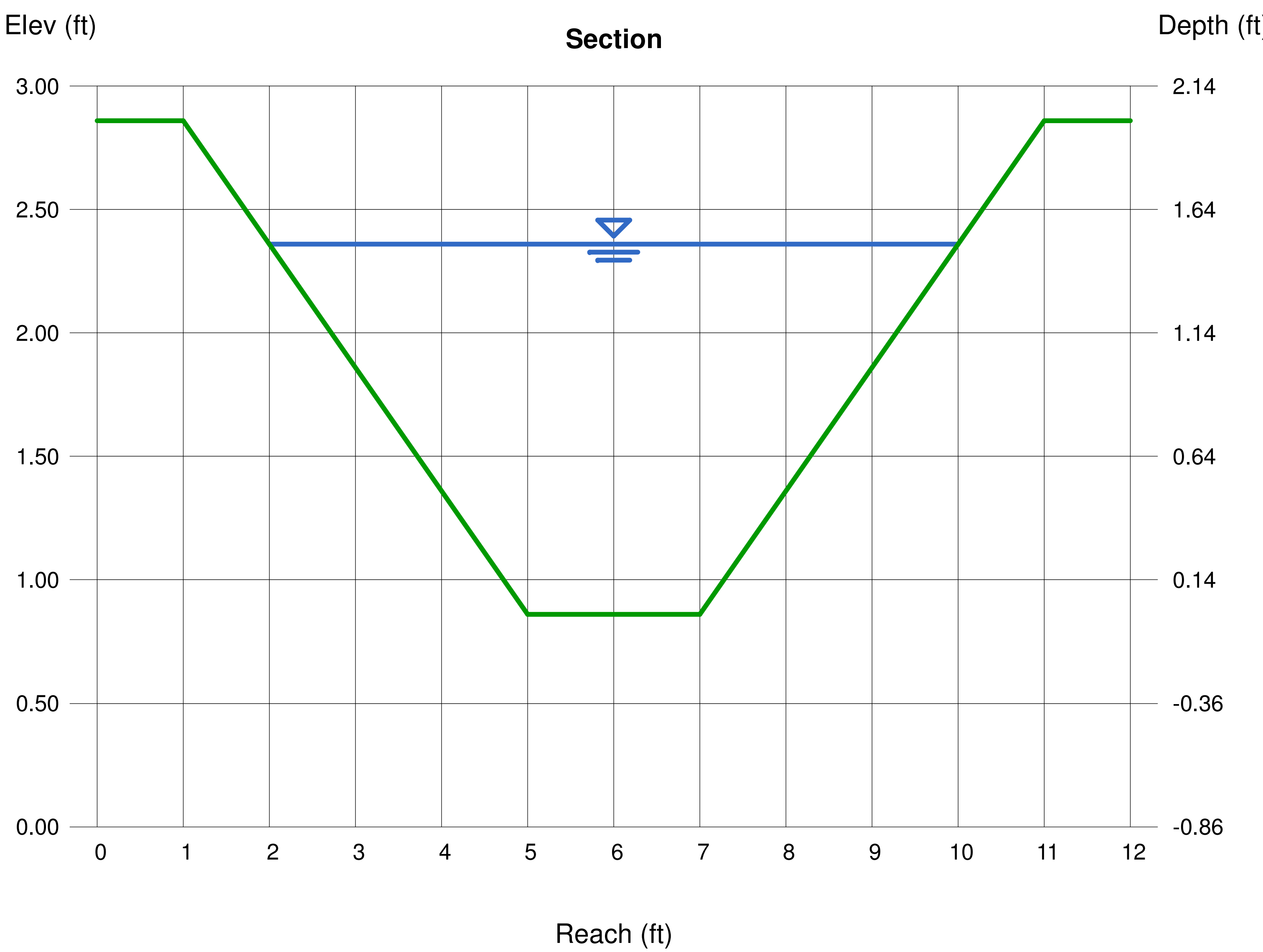
Channel Report

Hydraflow Express by Intelisolve — Friday, Feb 20 2015

CONCEPT #3 OO_12/OO_13 - IMPROVED DITCH 1

Trapezoidal			**Highlighted**		
Botom Width (ft)	=	2.00	Depth (ft)	=	1.50
Side Slopes (z:1)	=	2.00, 2.00	Q (cfs)	=	23.78
Total Depth (ft)	=	2.00	Area (sqft)	=	7.50
Invert Elev (ft)	=	0.86	Velocity (ft/s)	=	3.17
Slope (%)	=	0.50	Wetted Perim (ft)	=	8.71
N-Value	=	0.030	Crit Depth, Yc (ft)	=	1.14
			Top Width (ft)	=	8.00
Calculations			EGL (ft)	=	1.66
Compute by:		Known Depth			
Known Depth (ft)	=	1.50			

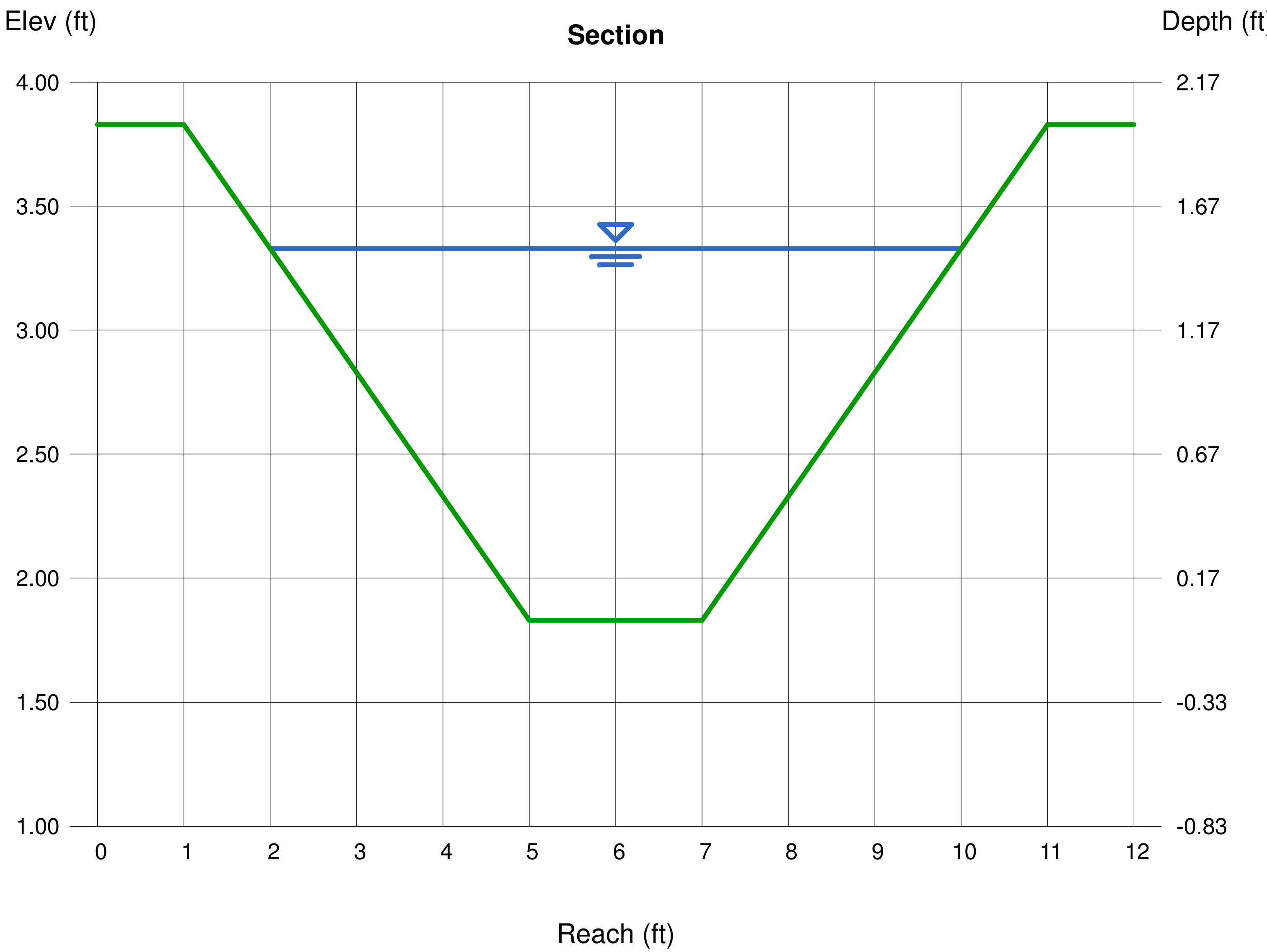


Channel Report

Hydraflow Express by Intelisolve | Friday, Feb 20 2015

CONCEPT #3 OO_12/OO_13 - IMPROVED DITCH 2

Trapezoidal			**Highlighted**		
Botom Width (ft)	=	2.00	Depth (ft)	=	1.50
Side Slopes (z:1)	=	2.00, 2.00	Q (cfs)	=	23.78
Total Depth (ft)	=	2.00	Area (sqft)	=	7.50
Invert Elev (ft)	=	1.83	Velocity (ft/s)	=	3.17
Slope (%)	=	0.50	Wetted Perim (ft)	=	8.71
N-Value	=	0.030	Crit Depth, Yc (ft)	=	1.14
			Top Width (ft)	=	8.00
Calculations			EGL (ft)	=	1.66
Compute by:		Known Depth			
Known Depth (ft)	=	1.50			



Culvert Report

Hydraflow Express by Intelisolve — Friday, Feb 20 2015

CONCEPT #3 OO_12/OO_13 - PR CULVERT 1

Invert Elev Dn (ft)	=	0.86
Pipe Length (ft)	=	27.91
Slope (%)	=	0.50
Invert Elev Up (ft)	=	1.00
Rise (in)	=	18.0
Shape	=	Cir
Span (in)	=	18.0
No. Barrels	=	1
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

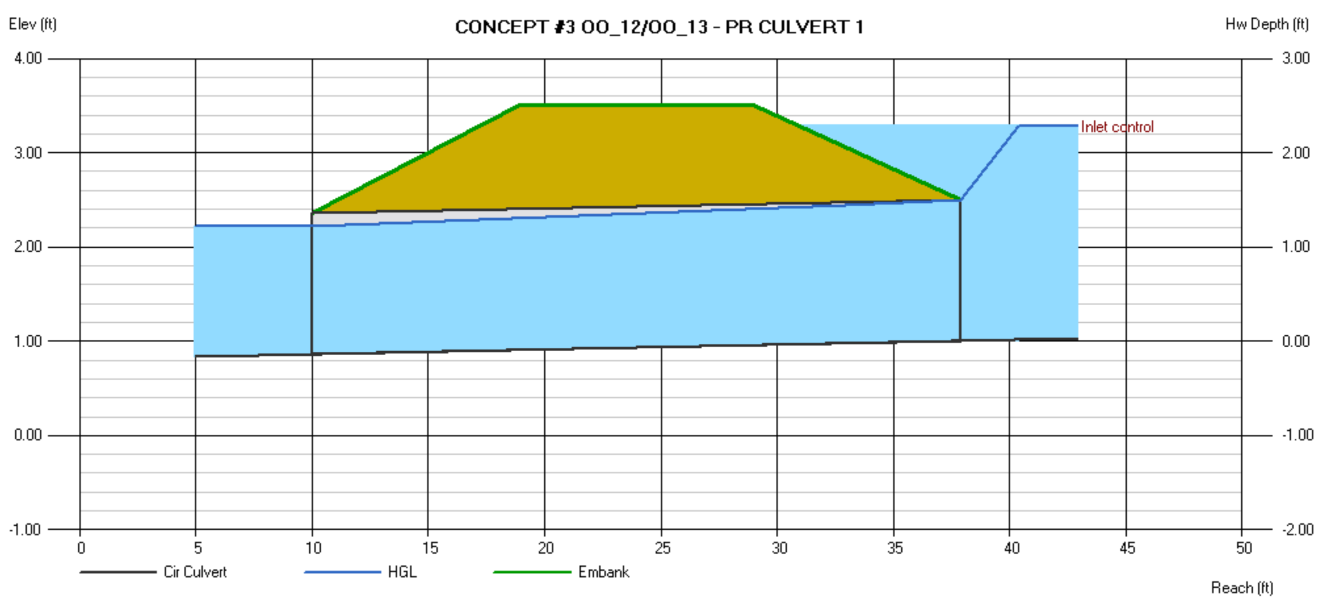
Top Elevation (ft)	=	3.50
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	10.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	10.00
Qpipe (cfs)	=	10.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	5.94
Veloc Up (ft/s)	=	5.66
HGL Dn (ft)	=	2.22
HGL Up (ft)	=	2.50
Hw Elev (ft)	=	3.28
Hw/D (ft)	=	1.52
Flow Regime	=	Inlet Control



Culvert Report

CONCEPT #3 OO_12/OO_13 - PR RCP-1

Invert Elev Dn (ft)	=	0.01
Pipe Length (ft)	=	109.52
Slope (%)	=	0.50
Invert Elev Up (ft)	=	0.56
Rise (in)	=	18.0
Shape	=	Cir
Span (in)	=	18.0
No. Barrels	=	1
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

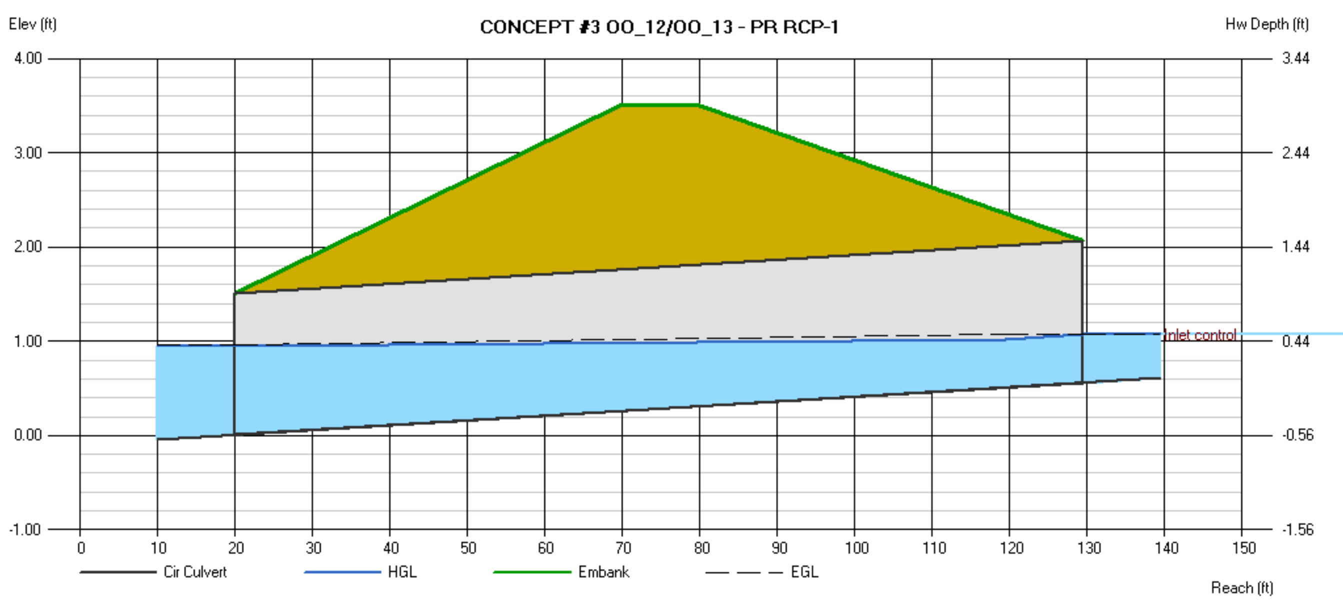
Top Elevation (ft)	=	3.50
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	10.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	1.00
Qpipe (cfs)	=	1.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	0.86
Veloc Up (ft/s)	=	2.19
HGL Dn (ft)	=	0.95
HGL Up (ft)	=	1.02
Hw Elev (ft)	=	1.07
Hw/D (ft)	=	0.34
Flow Regime	=	Inlet Control



Culvert Report

Hydraflow Express by Intelisolve — Friday, Feb 20 2015

CONCEPT #3 OO_12/OO_13 - PR RCP-2

Invert Elev Dn (ft)	=	0.01
Pipe Length (ft)	=	30.35
Slope (%)	=	0.49
Invert Elev Up (ft)	=	0.16
Rise (in)	=	18.0
Shape	=	Cir
Span (in)	=	18.0
No. Barrels	=	1
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

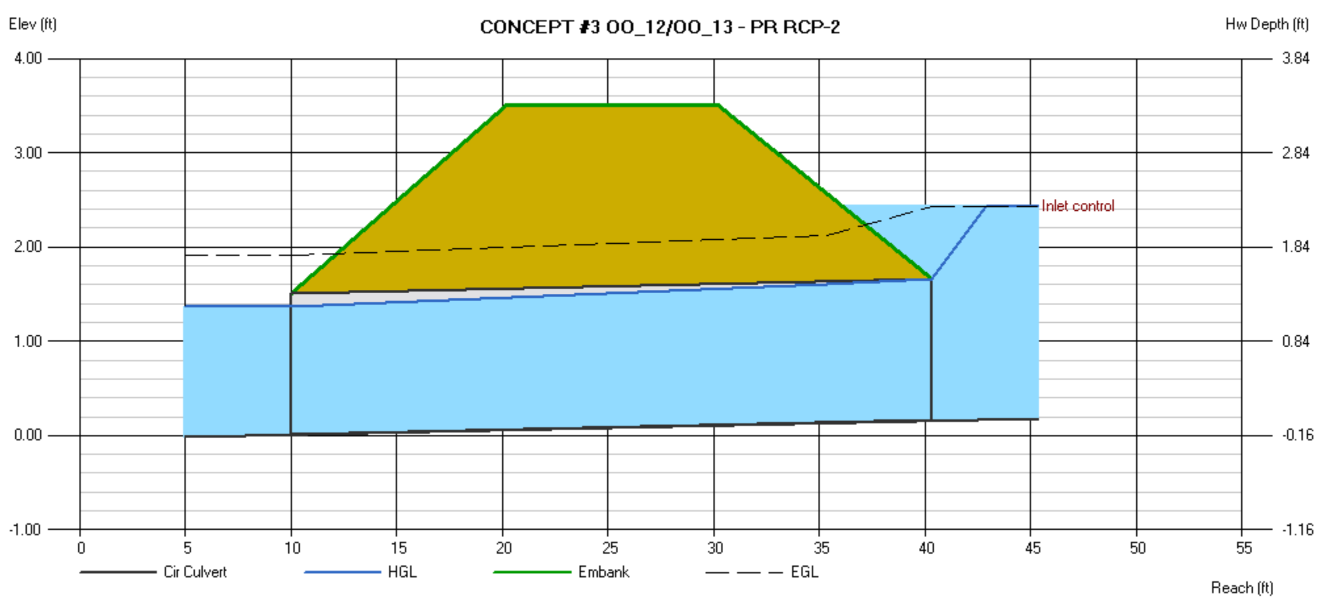
Top Elevation (ft)	=	3.50
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	10.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	10.00
Qpipe (cfs)	=	10.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	5.94
Veloc Up (ft/s)	=	5.66
HGL Dn (ft)	=	1.37
HGL Up (ft)	=	1.66
Hw Elev (ft)	=	2.44
Hw/D (ft)	=	1.52
Flow Regime	=	Inlet Control



Culvert Report

CONCEPT #3 OO_12/OO_13 - PR RCP-3

Invert Elev Dn (ft)	=	0.01
Pipe Length (ft)	=	71.62
Slope (%)	=	0.50
Invert Elev Up (ft)	=	0.37
Rise (in)	=	18.0
Shape	=	Cir
Span (in)	=	18.0
No. Barrels	=	1
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

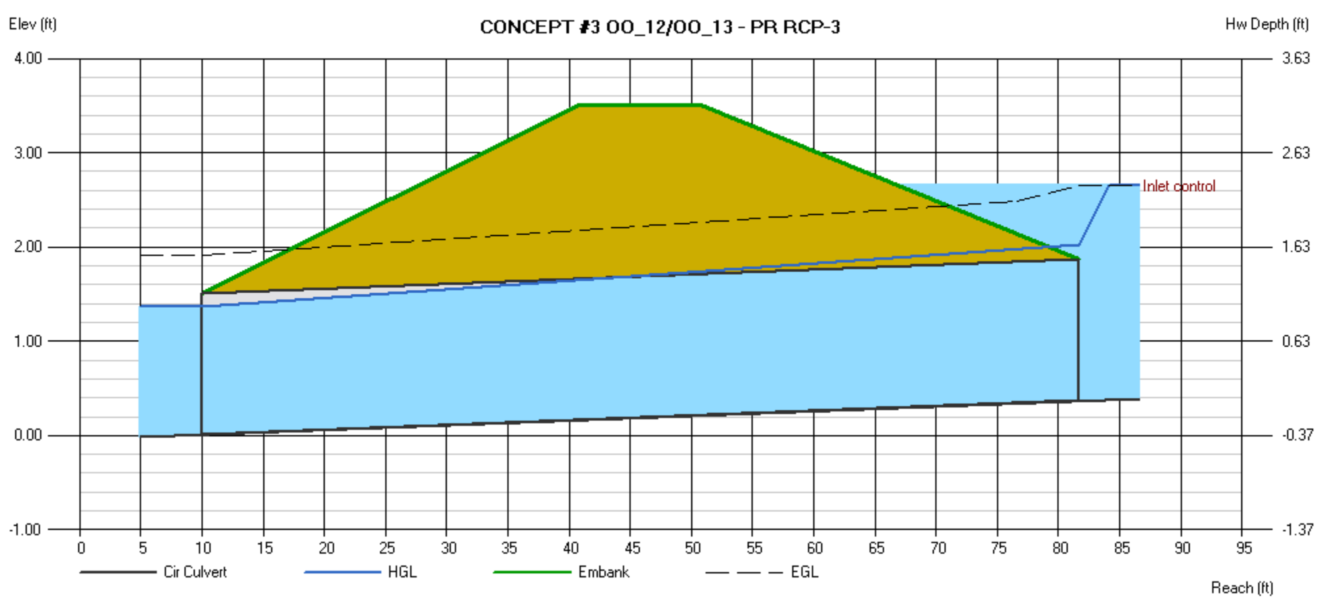
Top Elevation (ft)	=	3.50
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	10.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	10.00
Qpipe (cfs)	=	10.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	5.94
Veloc Up (ft/s)	=	5.66
HGL Dn (ft)	=	1.37
HGL Up (ft)	=	2.03
Hw Elev (ft)	=	2.65
Hw/D (ft)	=	1.52
Flow Regime	=	Inlet Control



Culvert Report

Hydraflow Express by Intelisolve | Friday, Feb 20 2015

CONCEPT #3 OO_12/OO_13 - PR RCP-4

Invert Elev Dn (ft)	=	0.06
Pipe Length (ft)	=	19.94
Slope (%)	=	0.50
Invert Elev Up (ft)	=	0.16
Rise (in)	=	18.0
Shape	=	Cir
Span (in)	=	18.0
No. Barrels	=	1
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

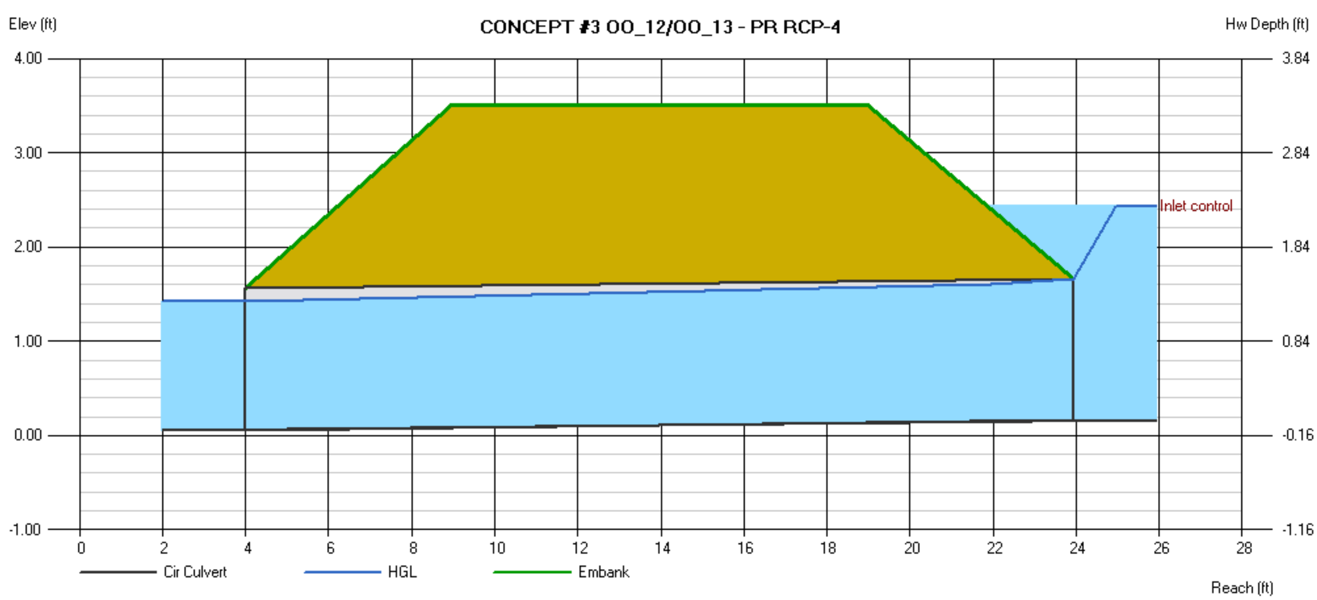
Top Elevation (ft)	=	3.50
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	10.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	10.00
Qpipe (cfs)	=	10.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	5.94
Veloc Up (ft/s)	=	5.69
HGL Dn (ft)	=	1.42
HGL Up (ft)	=	1.62
Hw Elev (ft)	=	2.44
Hw/D (ft)	=	1.52
Flow Regime	=	Inlet Control



Culvert Report

CONCEPT #3 OO_12/OO_13 - PR RCP-5

Invert Elev Dn (ft)	=	0.16
Pipe Length (ft)	=	20.62
Slope (%)	=	0.48
Invert Elev Up (ft)	=	0.26
Rise (in)	=	18.0
Shape	=	Cir
Span (in)	=	18.0
No. Barrels	=	1
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

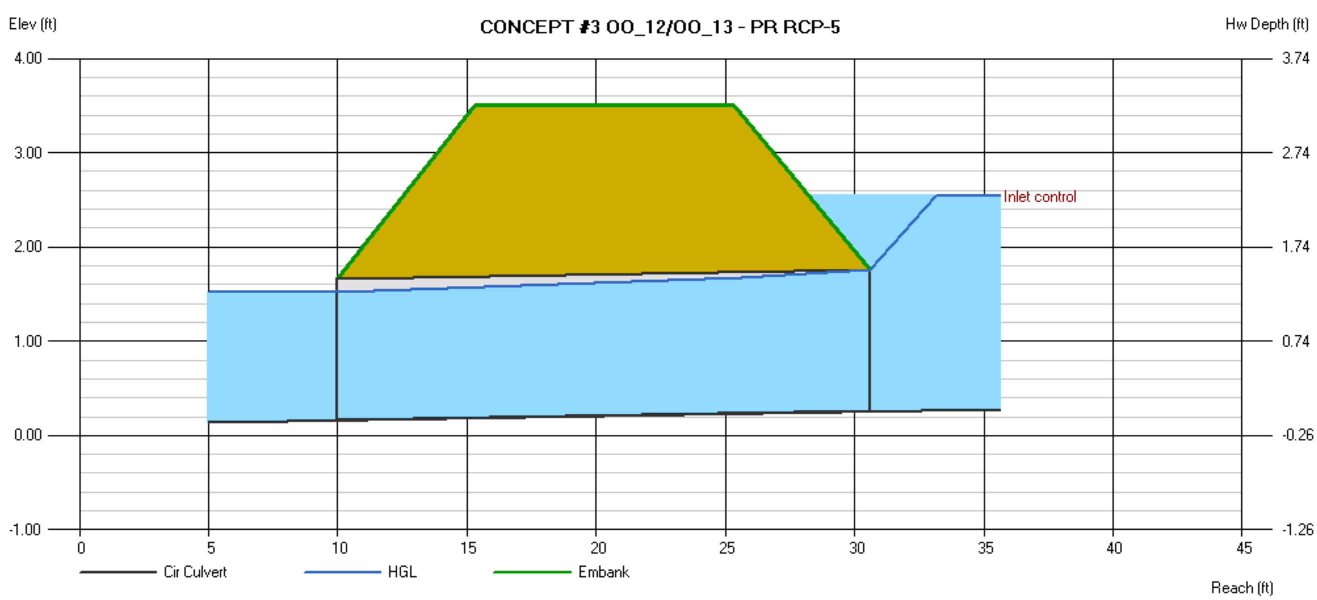
Top Elevation (ft)	=	3.50
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	10.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	10.00
Qpipe (cfs)	=	10.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	5.94
Veloc Up (ft/s)	=	5.68
HGL Dn (ft)	=	1.52
HGL Up (ft)	=	1.73
Hw Elev (ft)	=	2.54
Hw/D (ft)	=	1.52
Flow Regime	=	Inlet Control



Concept Design OO_18

1 EXISTING SITE DESCRIPTION

The questionnaire responses indicate that water collects at the western end of Fairfax Court and James Court in the Captain's Grant development. This appears to be caused by the lack of clear conveyances toward the roadside channel along Oak Orchard Road and results in ponding and backing up of driveway culverts several times a year. Oak Orchard Road also ponds near this location, suggesting the roadside channel is inadequate to convey the cumulative runoff.

Channel at the end of West Fairfax Court. Sediment is preventing conveyance of water from higher areas.

James and Fairfax Courts are both cul-de-sacs sloping from east to west. Each is intended to drain by surface swale to a channel along the eastern side of Oak Orchard Road. There is a well-defined channel at the end of Fairfax Court, but excessive sedimentation appears to be causing backups. There is less defined swale at the end of James Court. Furthermore, the roadside channel along Oak Orchard Road becomes less and less discernable as it progresses upstream in a southerly direction.

Roadside channel along Oak Orchard Road downstream of West James Court and West Fairfax Court.

Farther downstream, the roadside channel flows in a northerly direction and ends at Captain's Way, where flow is conveyed under the road by a 15-inch culvert to another channel that flows into a large stormwater management wet pond. The outfall from this basin flows to an inlet on the east side of Oak Orchard Road and then under the road to a very slow-moving stream. This culvert was totally submerged during our February 2015 field investigation, and its size is unknown.

Figure 1 and the photographs show the existing conditions.

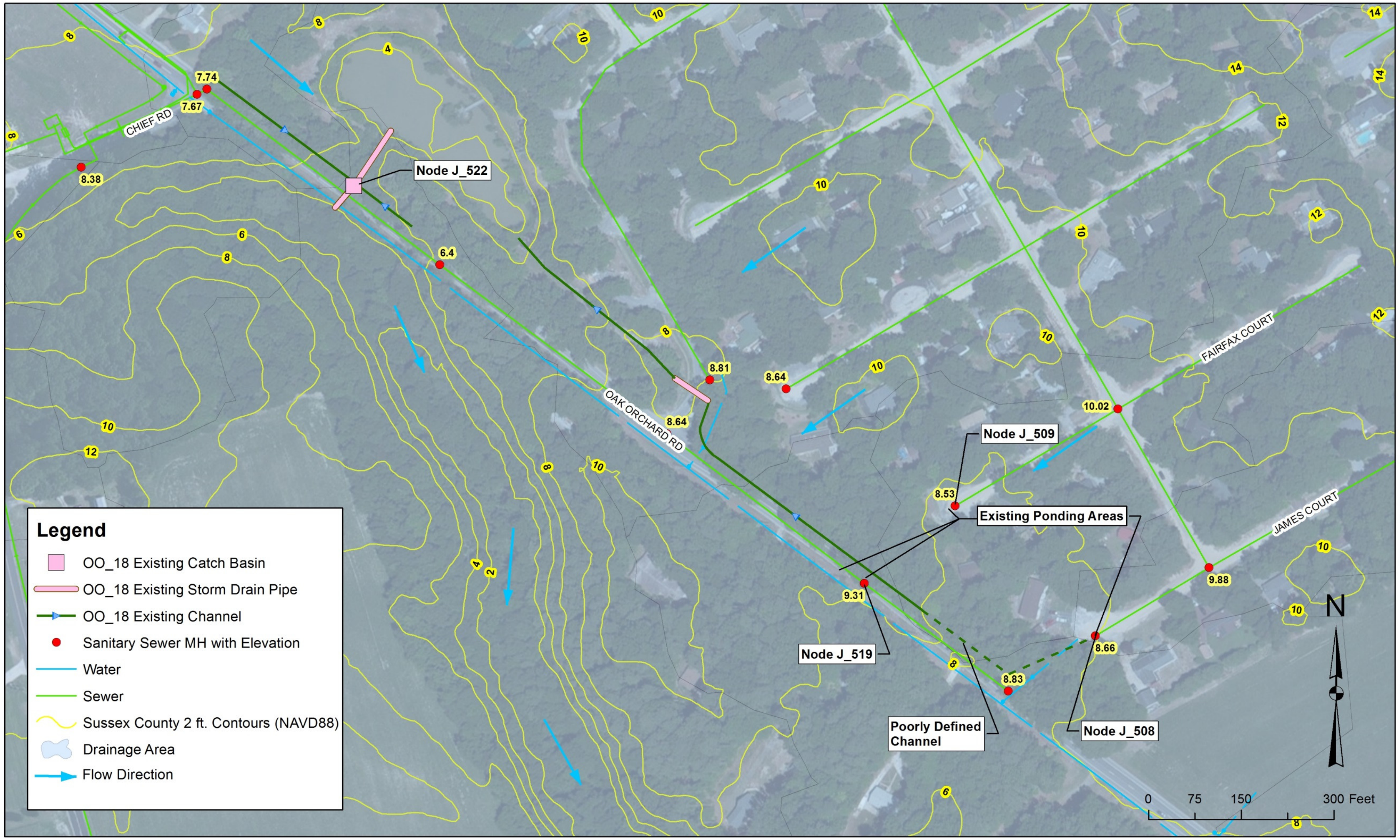


Figure 1: OO_18 Existing Site Conditions

2 PROPOSED IMPROVEMENT

The proposed design at this location includes both maintenance and construction tasks. The channel from Fairfax Court is generally well defined but is clogged with sediment. The Oak Orchard Road roadside channel from this point to Captain's Way likewise is well defined, but some sediment is present. In both cases sediment should be removed and the channels regraded.

Low channel velocities make it difficult for open channels to self-maintain, resulting in periodic maintenance needs.

The roadside channel upstream of the convergence with the Fairfax Court channel is not well defined, nor is the channel from James Court. In both these cases, new channels are needed. Also, the roadside channel should be extended farther to the south to help drain ponded areas south of the Captain's Grant development off Oak Orchard Road.

The culvert under Captain's Way should be increased in size to 24 inches in diameter. However, the channel from this location to the existing stormwater management basin appears to be adequate. The outlet from the basin flows to an inlet off the eastern shoulder of Oak Orchard Road, and from there, a culvert runs under Oak Orchard Road. The inlet is problematic. About a foot of leaves and debris was found on its top during a February 2015 site visit. The Delaware Department of Transportation (DelDOT) will need to be consulted regarding options, which may include replacing the horizontal grate with more of a "rooftop" configuration or installing an open-throated inlet, neither of which would be as prone to clogging. Another approach would be to construct a sediment trapping device, potentially in conjunction with raising the grate elevation, although this could result in ponding in the right-of-way.

The downstream end of the culvert under Oak Orchard Road was inundated during a February 2015 site visit, so increasing its size would do little to increase conveyance under the road. If this pipe were to be upsized, the downstream channel would also need to be cleared. In lieu of upsizing this culvert, it may be possible to reconfigure the stormwater management basin outlet by placing an improved riser or regrading the basin side slopes. As-built plans and design calculations were unavailable for this basin.

Reliance on surface drainage coupled with the wooded conditions of the Captain's Grant development will necessitate periodic future maintenance. Low velocities such as those found in relatively flat channels are unlikely to convey leaves downstream in autumn. Homeowners may need to clear the channels on or adjacent to their properties, and DelDOT may need to perform the same function along Oak Orchard Road. Easements may be needed for improvements to the channels at Fairfax and James Courts.

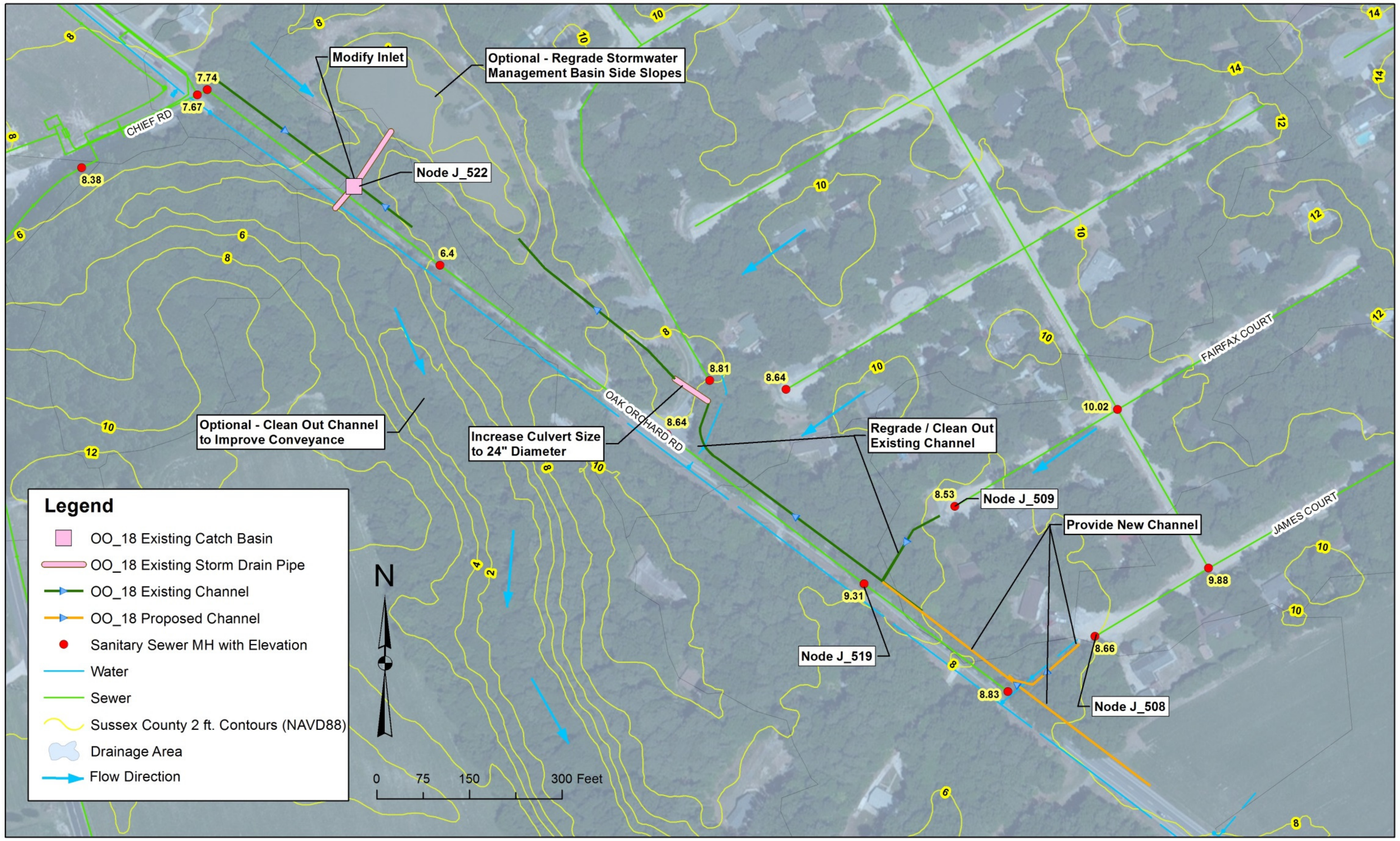


Figure 2: OO_18 Proposed Site Design

3 HYDRAULIC AND HYDROLOGIC CALCULATIONS

3.1 HEC-HMS Hydrologic Analysis

A hydrologic analysis for Oak Orchard was performed using HEC-HMS. The methodology and results of this study are discussed in Appendix F. The peak flows downstream of James Court (node J_508), downstream of Fairfax Court (node J_509), at the convergence of the two (node J_519), and at the discharge of the stormwater management basin (node J_522) are shown in Table 1.

Table 1 Peak Flow Rates

Name	Location Description	Drainage Area, mi^2	Storm Event Flows (cubic feet per second)				
			2-year	10-year	25-year	50-year	100-year
J_508	Flow downstream of James Court	0.02	4	10	15	20	26
J_509	Flow downstream of Fairfax Court	0.01	3	6	9	12	16
J_519	Combined flow from James and Fairfax Courts	0.03	7	16	24	31	40
J_522	Flow leaving the retention pond	0.07	6	26	41	54	70

3.2 Existing Hydraulic Calculations

The Inteli Solve HydraFlow Express (2006) program, which is based on Manning's equation, was used to model the storm drain, culverts, and channels using the 10-year peak flow rates calculated in HEC-HMS as described above. To use HydraFlow Express, hydrologic input files had to be developed to match those in the URS HEC-HMS study for each catchment area using drainage area, time of concentration, and curve number. Peak flows were calculated at known points in the storm network.

3.3 Proposed Hydraulic Calculations

The proposed hydraulic conditions were calculated using the same method used for the existing hydraulic conditions. HydraFlow Express was used to size proposed storm drains, culverts, and channels to meet the permissible velocity and freeboard requirements specified in the DelDOT Road Design Manual (2008). A 10-year design storm was used as the basis for our evaluations as specified in the manual for storm drain, culverts, and roadside channels along local roads and rural collectors.

An iterative approach had to be used to calculate the required pipe and channel sizes because of limited topographic relief throughout site and other site constraints. The channels should be constructed or redefined to the following trapezoidal dimensions:

- Bottom Width = 2 feet
- Side Slopes = 2H:1V
- Total Depth = 2 feet
- Slope = 0.5 %

The existing culvert under Captain's way should be replaced with a 24-inch reinforced concrete pipe or high-density polyethylene pipe. This pipe could be either reinforced concrete or high-density polyethylene. The cost estimate in Section 7 is based on polyethylene. Material selection would be made by DelDOT.

4 IMPROVEMENTS AND BENEFITS

The proposed design would decrease the frequency and duration of ponding from localized runoff at Fairfax and James Courts, as well as Oak Orchard Road. Grading in new channels and clearing existing channels will expedite the removal of runoff from the site by allowing the channels to flow at design capacity. Replacing the existing pipe under Captain's Way will provide the additional capacity needed to carry runoff from the ponding areas to the existing stormwater management pond. Modifying the inlet immediately downstream from the pond would reduce its tendency to clog with leaves and debris. If the channel downstream of the pipe leading from this inlet were cleared, additional flow may able to be attained through the pipe under Oak Orchard Road.

5 FEASIBILITY ASSESSMENT

Soil and Groundwater: The soils in the drainage area are all Fort Mott loamy sand with 0 to 2 percent slopes (NRCS). Fort Mott loamy sand is classified as hydrologic group A, which are well drained soils primarily composed of sand. Sandy soils have no cohesion, so design velocities will need to be considered carefully during final design to avoid erosion. Groundwater data from the Delaware Geological Survey suggest that the water table is approximately 7 to 10 feet below the ground surface. Field exploration would be needed to confirm this. If actual elevations are higher, it could result in standing water in both the existing and proposed channels. In these situations, the channels would convey less water after precipitation events, but the design is still expected to expedite the removal of runoff from the site.

Construction Access: These sites are easily accessible from both cul-de-sacs and Oak Orchard Road, but work between the cul-de-sacs and Oak Orchard Road would necessitate easements if none already exists. Construction equipment may need to be parked on the cul-de-sacs or Captain's Way.

Maintenance Considerations: Routine maintenance will be required to sustain the designed channel flow capacity. Maintenance would include periodically removing sediment and leaves and cutting / removing grass. Though phragmites were not observed, any that do emerge would need to be removed as well. Routine maintenance will also be needed to keep the inlet to the north of Captain's Way open.

Utility Conflicts: A sanitary sewer exists along the eastern side of Oak Orchard Road, but since proposed improvements are only to the surface, conflicts should not develop. Water lines exist more or less coincident with the existing channels from the two cul-de-sacs. Ideally these water mains would be relocated. Locations of any other below-ground utilities such as water or gas or above-ground utilities such as electric lines would need to be confirmed during detailed design.

Effectiveness: The proposed design would substantially reduce nuisance ponding from frequent storm events. Ponding from large coastal events would still be expected; however, the duration of ponding should be reduced. The effectiveness of the proposed design would be heavily dependent on the routine maintenance of the proposed channels.

Environmental Issues: Clearing the channel downstream of the stormwater management basin outlet pipe could affect wetlands if present. Remaining construction and maintenance would occur in areas already developed. Existing trees south of these two cul-de-sacs may impede the proposed channel system and may require removal.

Easements: Easements may be needed for improvements to the channels at Fairfax and James Courts if none already exist.

6 PLANS AND PERMITTING

Several construction documents and plans would need to be obtained to implement the proposed drainage design, including, but not limited to those described in Table 2.

Table 2: Required Plans and Permitting for Proposed Design OO_18

Plans/Permits	Permitting Agency	Notes and Potential Difficulties
Wetlands and Subaqueous Lands Permit	DNREC	The existing channel downstream of the stormwater management basin outlet pipe may be hydraulically connected to wetlands farther downstream.
Traffic Control Plan	DelDOT	
Erosion and Sediment Control Plan	Sussex Conservation District	
Utility Construction Permit	DelDOT	Limited utility impacts are anticipated for this project.

7 COST ESTIMATE

Table 3 summarizes the costs associated with this concept design.

Table 3 Estimated Project Costs for OO_18

ITEM	QUANTITY	UNITS	UNIT COST	TOTAL
Excavation for Proposed Channels	70	CY	$60.00	$4,200
Grading	600	SY	$2.50	$1,500
Regrade Existing Channels	500	LF	$10.00	$5,000
Seeding and Mulching	1150	SY	$2.50	$2,875
Remove and Dispose Piping	40	LF	$20.00	$800
Asphalt Base	9	TON	$100.00	$900
Asphalt Surface	5	TON	$110.00	$550
Reinforced Concrete Pipe Culvert	30	LF	$100.00	$3,000
Riprap	10	SY	$90.00	$900
Inlet	1	EA	$5,000.00	$5,000
Traffic Control Plan	5	DAY	$750.00	$3,750
New Riser at Stormwater Management Basin	1	EA	$10,000.00	$10,000

CY = cubic yard
EA = each
LF = linear foot
SY = square yard

Initial Project Costs		$38,475
Contingency 10%		$3,848
Erosion and Sediment Control	10%	$3,848
Base Construction Costs		$46,170
Mobilization	5%	$2,309
Subtotal 1		$48,479
Contingency	15%	$7,272
Subtotal 2		$55,750
Engineering		$20,000
Total		$75,750[a]

[a] Based on field location of existing utilities, relocation of the existing sewer line may be required, however these costs are not included.

8 REFERENCES

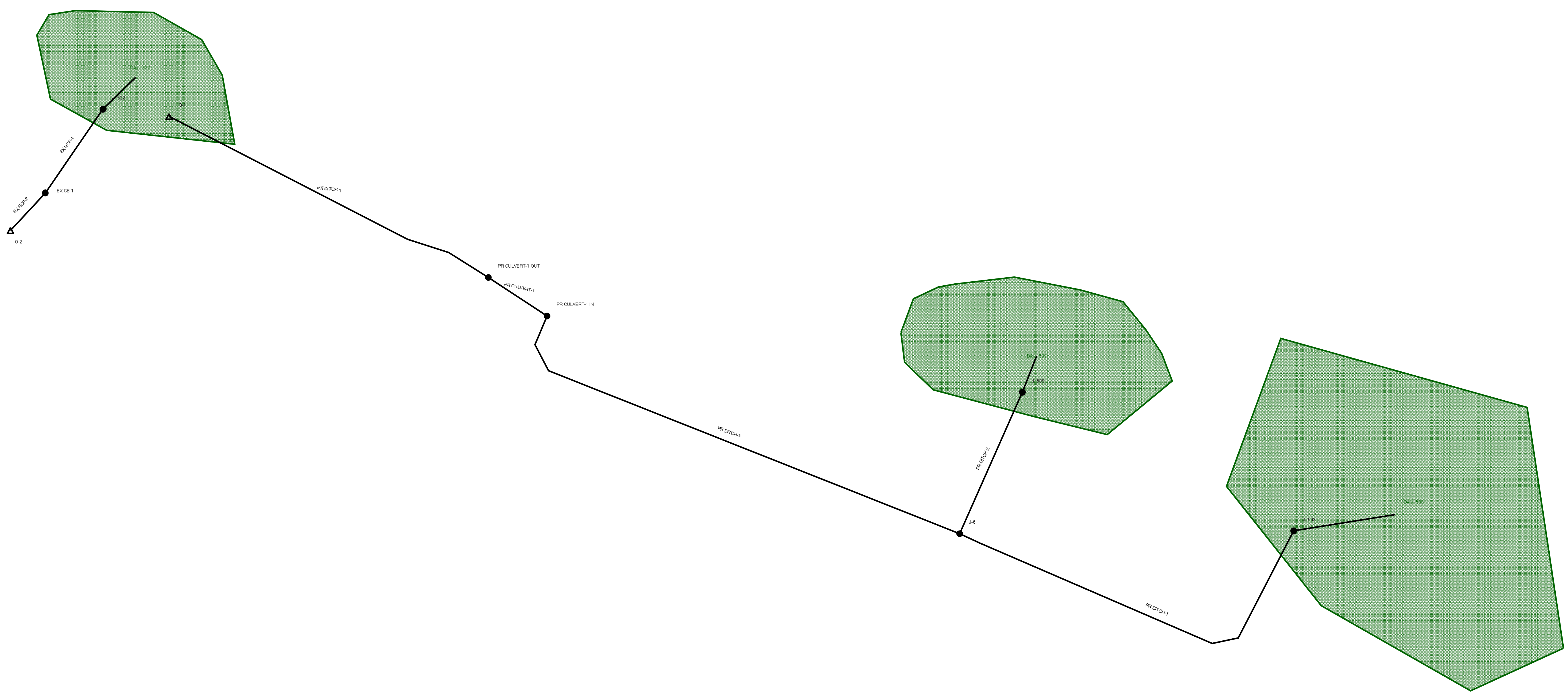
Delaware Department of Transportation (DelDOT), 2008. Road Design Manual Chapter Six: Drainage and Stormwater Management. Available at http://www.deldot.gov/information/pubs_forms/manuals/road_design/pdf/06_drainage_stormwater_mgmt.pdf

Intelisolve Hydraflow Express Version 1.08. Available at http://intelisolve.software.informer.com/

Water Table in the Inland Bays Watershed, Delaware. 2005. Delaware Geological Survey (DGS), accessed February 2015. Available at http://www.dgs.udel.edu/.

NRCS, 2009. U.S. General Soil Survey Geographic Database (SSURGO), accessed November 2014. Available at http://SoilDataMart.nrcs.usda.gov/.

Scenario: 10 Year



CONCEPT DESIGN #4_ OO_18 (CAPTAINS GRANT)

Project Summary	
Title	
Engineer	
Company	
Date	2/9/2015

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CONCEPT DESIGN #4_ OO_18 (CAPTAINS GRANT)

Subsection: Modified Rational Grand Summary

Modified Rational Method

Q = CiA * Units Conversion; Where conversion = 43560 / (12 * 3600)

Frequency (years)	Area ($mile^2$)	Adjusted C Coefficient	Duration (hours)	Intensity (in/h)	Flow (Peak) (ft^3/s)	Flow (Allowable) (ft^3/s)	Volume (inflow) (ac-ft)	Volume (Storage) (ac-ft)
10	0.020	1.000	4.500	0.800	10.33	2.84	3.840	2.787
10	0.010	1.000	3.500	0.927	5.98	1.42	1.730	1.320
10	0.010	1.000	0.350	4.056	26.17	1.42	0.757	0.716

CONCEPT DESIGN #4_ OO_18 (CAPTAINS GRANT)

Subsection: Master Network Summary

Catchments Summary

Label	Scenario	Return Event (years)	Hydrograph Volume (ac-ft)	Time to Peak (hours)	Peak Flow (ft³/s)
DA-J_508	10 Year	10	3.840	0.100	10.33
DA-J_509	10 Year	10	1.730	0.100	5.98
DA-J_522	10 Year	10	0.757	0.100	26.17

Node Summary

Label	Scenario	Return Event (years)	Hydrograph Volume (ac-ft)	Time to Peak (hours)	Peak Flow (ft³/s)
EX CB-1	10 Year	10	0.838	0.050	4.91
J-6	10 Year	10	5.570	0.900	16.31
J_508	10 Year	10	3.840	0.100	10.33
J_509	10 Year	10	1.730	0.100	5.98
J_522	10 Year	10	0.757	0.100	26.17
O-1	10 Year	10	5.635	0.250	17.33
O-2	10 Year	10	0.859	0.100	4.91
PR CULVERT-1 IN	10 Year	10	5.570	1.050	16.31
PR CULVERT-1 OUT	10 Year	10	5.635	0.200	17.17

CONCEPT DESIGN #4_ OO_18 (CAPTAINS GRANT)

Subsection: I-D-F Table

Return Event: 10 years

Label: Rainfall Intensity Sussex County

Storm Event: Rainfall Intensity Sussex County - 10 Year

I-D-F Curve

Time (hours)	Intensity (in/h)
0.083	6.760
0.167	5.400
0.250	4.560
0.500	3.300
1.000	2.150
2.000	1.350
3.000	0.990
6.000	0.610
12.000	0.360
24.000	0.220

CONCEPT DESIGN #4_ OO_18 (CAPTAINS GRANT)

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Channel Report

CONCEPT #4 OO_18 - IMPROVED DITCH 1

Trapezoidal

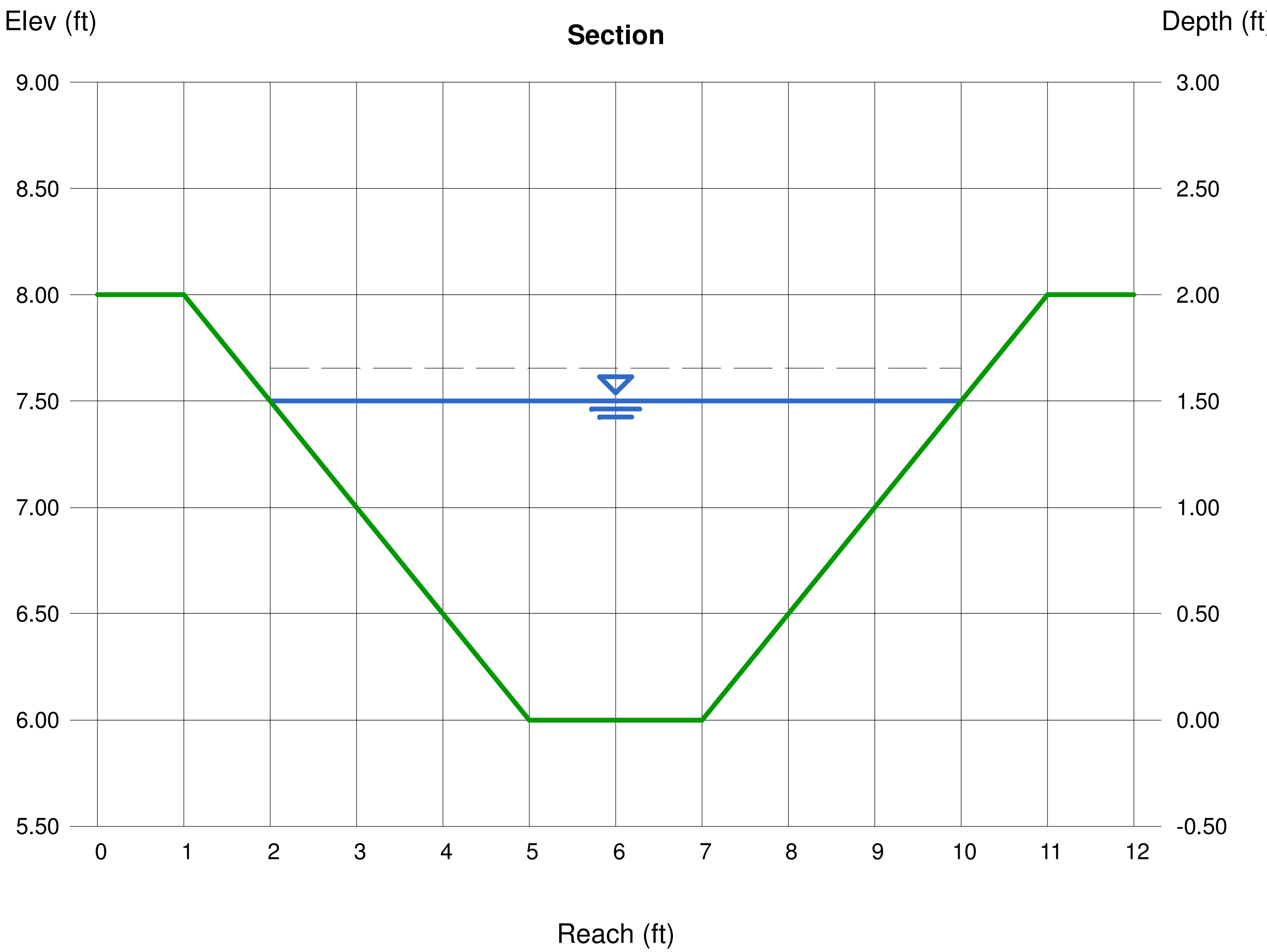
Botom Width (ft)	= 2.00
Side Slopes (z:1)	= 2.00, 2.00
Total Depth (ft)	= 2.00
Invert Elev (ft)	= 6.00
Slope (%)	= 0.50
N-Value	= 0.030

Calculations

Compute by:	Known Depth
Known Depth (ft)	= 1.50

Highlighted

Depth (ft)	= 1.50
Q (cfs)	= 23.78
Area (sqft)	= 7.50
Velocity (ft/s)	= 3.17
Wetted Perim (ft)	= 8.71
Crit Depth, Yc (ft)	= 1.14
Top Width (ft)	= 8.00
EGL (ft)	= 1.66



Culvert Report

CONCEPT #4 OO_18 - PR CULVERT-1

Invert Elev Dn (ft)	=	1.57
Pipe Length (ft)	=	62.48
Slope (%)	=	0.50
Invert Elev Up (ft)	=	1.88
Rise (in)	=	18.0
Shape	=	Cir
Span (in)	=	18.0
No. Barrels	=	1
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

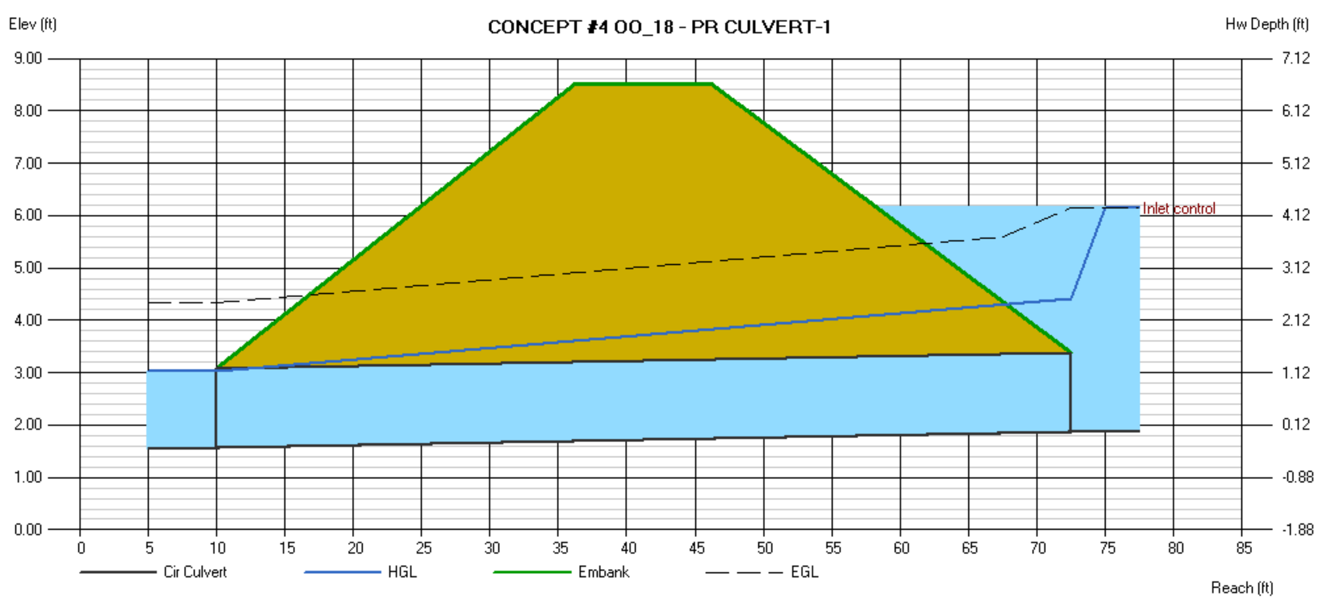
Top Elevation (ft)	=	8.50
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	20.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	16.00
Qpipe (cfs)	=	16.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	9.11
Veloc Up (ft/s)	=	9.05
HGL Dn (ft)	=	3.03
HGL Up (ft)	=	4.42
Hw Elev (ft)	=	6.14
Hw/D (ft)	=	2.84
Flow Regime	=	Inlet Control



Concept Design OO_22 / OO_28

1 EXISTING SITE DESCRIPTION

The questionnaire responses indicate that the southeastern corner of Oak Meadow Drive and Briar Lane ponds monthly, and Thistle Lane, Clover Lane, and Briar Lane also pond near existing catch basins. During field investigations in September 2014 and February 2015, numerous pockets of ponded water were observed.

Intersection of Briar Lane with Oak Meadow Drive lacks discernable drainage conveyance, and therefore experiences significant ponding

The intent in the Oak Meadow neighborhood was for it to drain predominantly by open channels that convey stormwater to a storm sewer system running through approximately the center of the development. However, the channels in some locations are not well defined and are lacking in other locations. Where channels do exist, they appear to be functioning fairly well.

Channels to convey surface water to catch basins are nonexistent in some locations

The storm sewer system consists of 12- to 18-inch pipes, which is too small to adequately convey runoff from frequent rain events. Furthermore, several of the inlet pipes are at lower elevations than outlet pipes at catch basins, meaning these portions of the system will not drain by gravity. Finally, while there is several feet of fall between the upper end of the drainage system at Briar Lane and the lower end at the existing downstream wet pond between Amber Drive and Devon Drive in the Crossing at Oak Orchard development to the south, the length between the two results in fairly flat pipes. The existing discharge point into the wet pond was partially submerged during our field investigations. No headwall exists. The existing conditions are depicted in Figures 1 and 2.

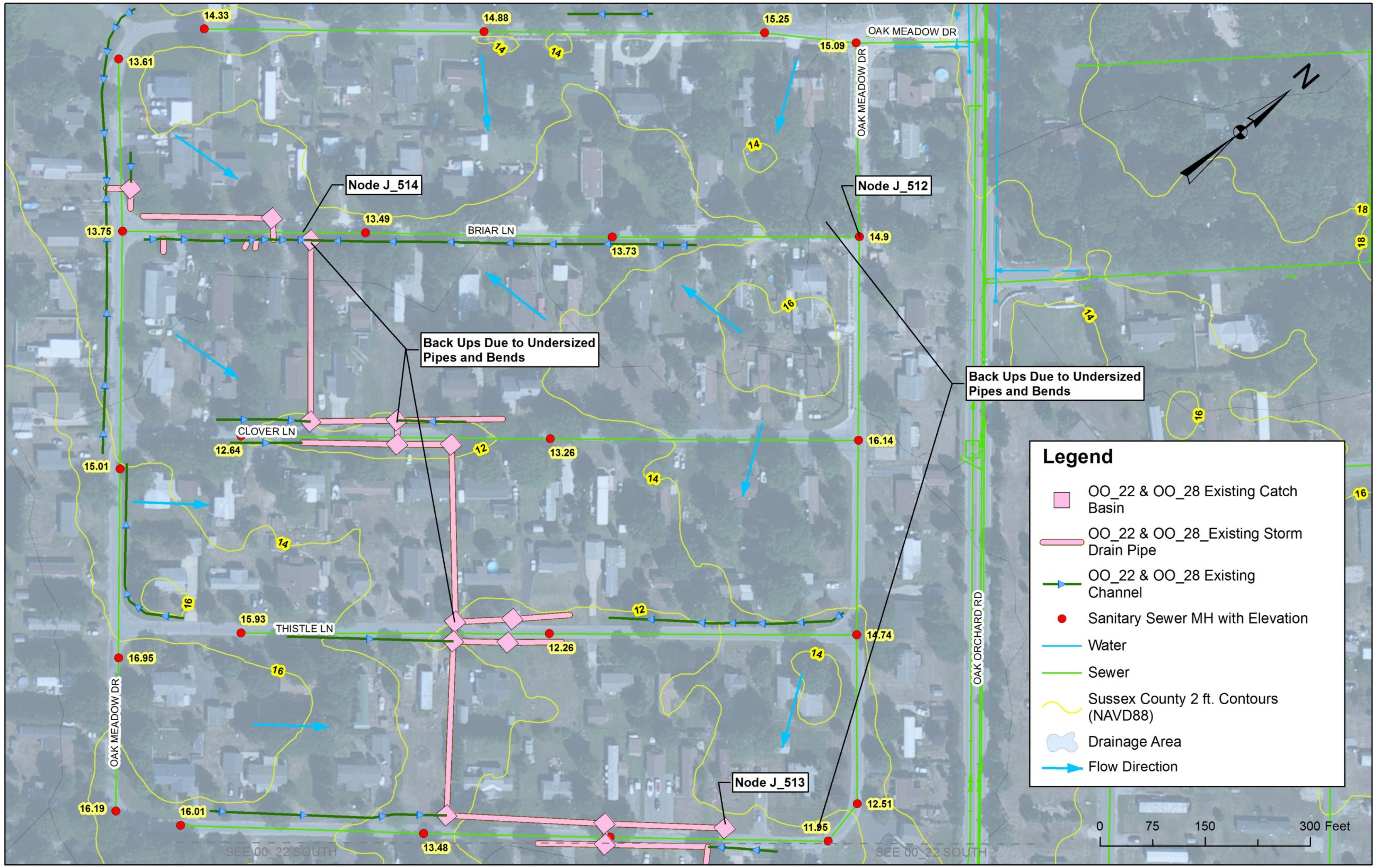


Figure 1: OO_22 / OO_28 North Existing Site Conditions

Figure 2: OO_22 / OO_28 South Existing Site Conditions

2 PROPOSED IMPROVEMENT

Constructing a new storm sewer system would improve surface drainage (top) and prevent driveway culverts from clogging (bottom)

The proposed design at site OO_22 / OO_28 is essentially a rebuild of the entire drainage system. Due to the distance between the top of the system at Briar Lane to its end at the existing wet pond in the Crossing at Oak Orchard development to the south, these pipes will have much less than the desired 0.5% slope. Initial calculations indicate that 36 inch twin pipes are required to convey the computed storm flows through Oak Meadow to the pond. A headwall should be added at the discharge point into the pond.

The proposed stormdrain alignment would also enable gravity flow at catch basins. Further, proposed realignment would eliminate the 90 degree bends through the Crossing at Oak Orchard to the existing downstream wet pond, to provide a more hydraulically efficient flow path. However, the existing storm sewer system crosses multiple yards, numerous easements would be required for this proposed alignment.

At least two new channels are also proposed. The first would run along the north side of Briar Lane from its intersection with Oak Meadow Drive to the enclosed drainage system. The second would be at the southeast corner of the development where Oak Meadow Drive makes a 90 degree bend. Existing channels that are not well defined should also be improved as the storm drain system is designed. These can either be identified as part of final design or field-identified and graded during construction.

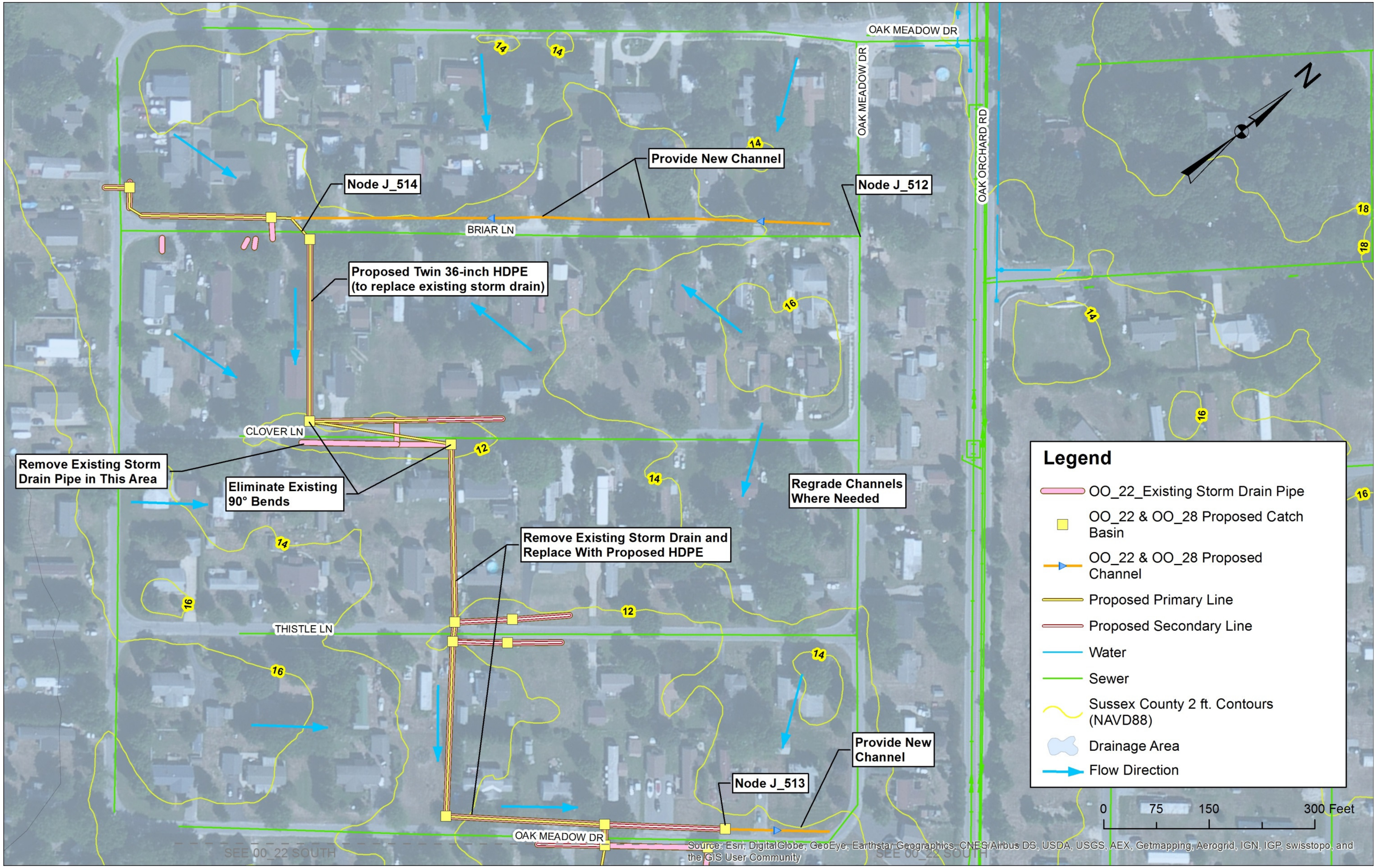


Figure 3: OO_22 / OO_28 North Proposed Site Design

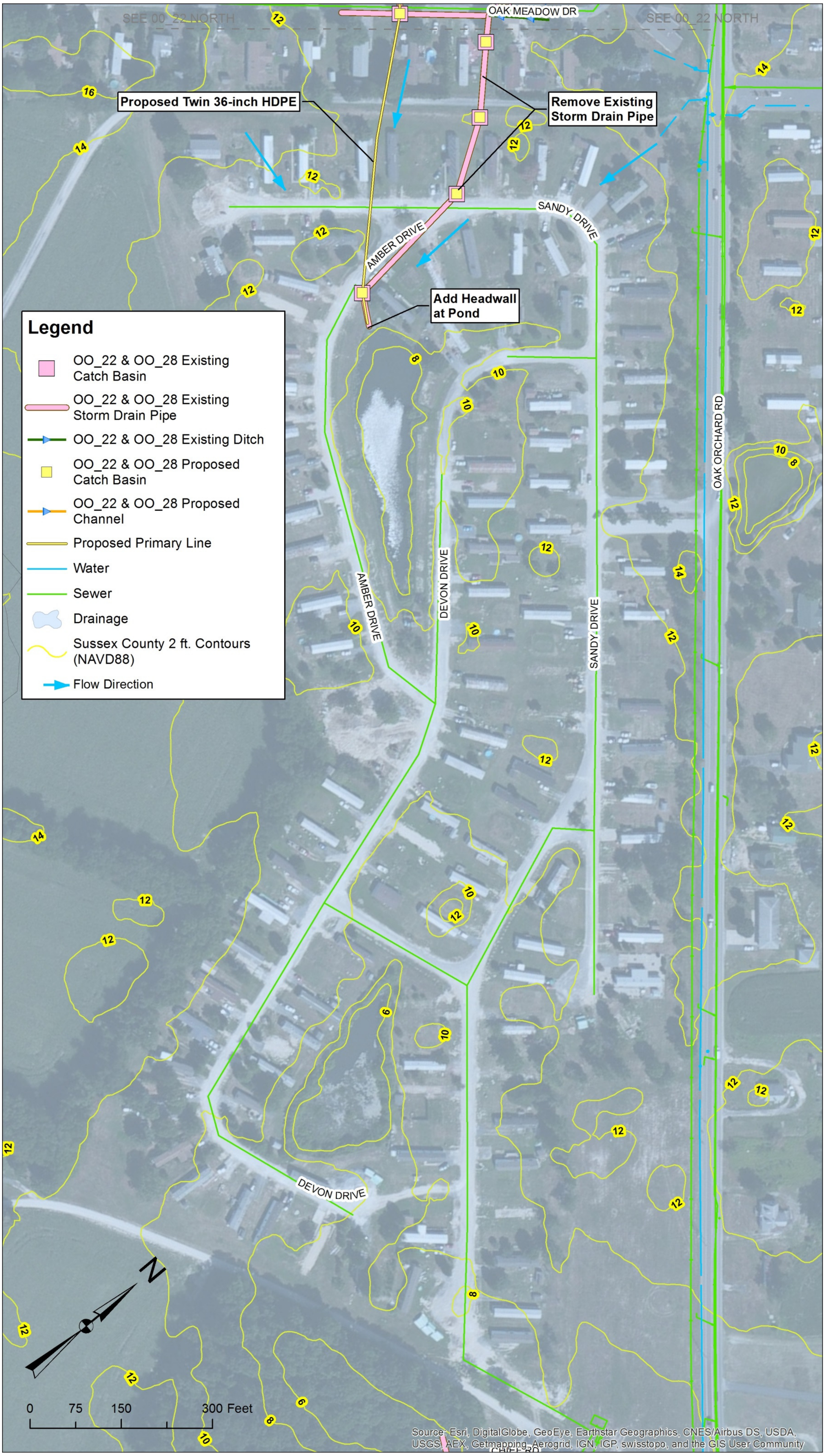


Figure 4: OO_22 / OO_28 South Proposed Site Design

3 HYDRAULIC AND HYDROLOGIC CALCULATIONS

3.1 HEC-HMS Hydrologic Analysis

A hydrologic analysis for Oak Orchard was performed using HEC-HMS. The methodology and results of this study are discussed in Appendix F. The peak flows at the intersection of Briar Lane and Oak Meadow Drive (node J_512), at the low spot in Briar Lane at the location of the existing and proposed inlets (node J_514), and where the existing and proposed conveyance system leaves Oak Meadow (node J_513) are shown in Table 1:

Table 1 Peak Flow Rates

Name	Location Description	Drainage Area, mi^2	Storm Event Flows (cubic feet per second)				
			2-year	10-year	25-year	50-year	100-year
J_512	OO_28, at the corner of Briar Lane and Oak Meadow Drive	0.02	4	9	13	17	21
J_514	The center of Briar Lane (at approximately the location of the storm drain pipe)	0.44	39	87	128	167	211
J_513	The stormdrain leaving Oak Meadow	0.48	42	91	134	174	220

3.2 Existing Hydraulic Calculations

The Inteli Solve HydraFlow Express (2006) program, which is based on Manning's equation, was used to model the storm drain, culverts, and channels using the 10-year peak flow rates calculated with HEC-HMS as described above. To use HydraFlow Express, hydrologic input files had to be developed to match those in the URS HEC-HMS study for each catchment area using drainage area, time of concentration, and curve number. Peak flows were calculated at known points in the storm network.

3.3 Proposed Hydraulic Calculations

The proposed hydraulic conditions were calculated using the same method used for the existing hydraulic conditions. HydraFlow Express was used to size proposed storm drains, culverts, and channels to meet the permissible velocity and freeboard requirements specified in the Delaware Department of Transportation (DelDOT) Road Design Manual (2008). A 10-year design storm was used as the basis for our evaluations as specified in the manual for storm drain, culverts, and roadside channels along local roads.

An iterative approach had to be used to calculate the required pipe and channel sizes because of limited topographic relief throughout site and other site constraints. The channels should be constructed or redefined to the following trapezoidal dimensions:

- Bottom Width = 2 feet
- Side Slopes = 2H:1V
- Total Depth = 2 feet
- Slope = 0.5%

The existing storm sewer should be replaced with twin 36-inch reinforced concrete pipe or high-density polyethylene pipes at the maximum slope existing grades allow.

4 IMPROVEMENTS AND BENEFITS

The proposed design would decrease the frequency and duration of ponding from localized runoff. Adding the proposed channels along Briar Lane and Oak Orchard Road and improving existing channels will expedite the removal of runoff from the site. Removal and replacement of the existing pipe system will provide the additional capacity needed to carry uphill runoff as well as runoff from the ponding areas not currently draining to this system.

5 FEASIBILITY ASSESSMENT

Soil and Groundwater: The soils at the proposed design and the drainage area are Pepperbox-Rosedale complex at 0 to 2 percent slopes and Fort Mott loamy sand at 2 to 5 percent slopes. Both of these soil types are classified as hydrologic soil group A, which are well-drained soils primarily composed of sand. Sandy soils have no cohesion, so design velocities will need to be considered carefully during final design to avoid erosion. Groundwater data from the Delaware Geologic Survey suggest that the water table is approximately 7 to 10 feet below the ground surface. Field explorations would be needed to confirm this. If actual elevations are higher, it could result in standing water in both the existing and proposed channels. In these situations, the channels would convey less water after precipitation events, but the design is still expected to expedite the removal of runoff from the site.

Construction Access: The lots at this site appear to be individually owned, but the manufactured homes at the Crossing at Oak Orchard to the south appear to be under common ownership. The grading of roadside channels would be easily accessible from public roads in Oak Meadow. The construction of a new storm sewer system, however, would necessitate work in multiple yards in Oak Meadow as well as those at the Crossing at Oak Orchard. Construction equipment may need to be parked along several roads.

Maintenance Considerations: Routine maintenance will be required to sustain the designed channel flow capacities. Maintenance would include periodically removing sediment and cutting grass in the channels. Though phragmites were not observed, any that do emerge would need to be removed as well. Routine maintenance will also be needed to keep inlets open and pipes clear.

Utility Conflicts: There is a sanitary sewer system in both developments. Several crossings of the storm sewer system already exist, and conflicts maybe developed if inverts are raised or lowered as part of the proposed design. Locations of any other below-ground utilities such as water or gas or above-ground utilities such as electric lines would need to be confirmed during detailed design.

Effectiveness: The proposed design would reduce nuisance ponding from frequent storm events. Ponding from large coastal events would still be expected; however, the duration of ponding should be reduced. The effectiveness of the proposed design would be dependent on the routine maintenance of the proposed channels and hydraulic improvements of the reconstructed storm sewer system.

Environmental Issues: Limited environmental impacts are expected from the earthmoving aspects of the proposed construction. However, this work would occur in areas already developed.

Easements: Since the existing storm sewer system crosses through multiple yards, numerous easements would be required for the reconstruction. For the areas where the storm sewer system is in the same location as the existing system, it is assumed that easements exist in these areas. The proposed project may require widening of the existing easements. For the two areas where proposed realignment of the storm sewer system is proposed, new easements will be required. If these easements cannot be obtained, the proposed storm sewer layout would need to be modified to be in the same location as the existing system.

6 PLANS AND PERMITTING

Several construction documents and plans would need to be obtained to implement the proposed drainage design, including, but not limited to, those described in Table 2

Table 2: Required Plans and Permitting for Proposed Design OO_22 / OO_28

Plans/Permits	Permitting Agency	Notes and Potential Difficulties
Wetlands and Subaqueous Lands Permit	DNREC	Work at the system outlet at the pond in the Crossing at Oak Orchard could necessitate a subaqueous permit.
Traffic Control Plan	DelDOT	Traffic control throughout Oak Meadow. Crossing at Oak Orchard appears to be privately owned.
Erosion and Sediment Control Plan	Sussex Conservation District	
Utility Construction Permit	DelDOT	Utilities throughout Oak Meadow. Crossing at Oak Orchard appears to be privately owned.

7 COST ESTIMATE

Table 3 summarizes the costs associated with this concept design.

Table 3 Estimated Project Costs for OO_22 / OO_28

ITEM	QUANTITY	UNITS	UNIT COST		TOTAL
Remove and Dispose Existing Piping	2500	LF	$20.00		$50,000
Excavation for Proposed Channels	225	CY	$60.00		$13,500
Grading	8000	SY	$2.50		$20,000
Seeding and Mulching	8000	SY	$2.50		$20,000
36" High Density Polyethylene Pipe	5000	LF	$80.00		$400,000
Inlets	20	EA	$2,500.00		$50,000
Headwall at Existing Pond	1	EA	$5,000.00		$5,000
Asphalt Base	100	TON	$100.00		$10,000
Asphalt Surface	50	TON	$110.00		$5,500
Traffic Control Plan	10	DAY	$750.00		$7,500
			Initial Project Costs		$581,500
			Contingency 10%		$58,150
		Erosion and Sediment Control		10%	$58,150
		Base Construction Costs			$697,800
		Mobilization		5%	$34,890
			Subtotal 1		$732,690
		Contingency		15%	$109,904
			Subtotal 2		$842,594
		Engineering			$75,000
				Total	$917,594[a]

CY = cubic yard
EA = each
LF = linear foot
SY = square yard

[a] Based on field location of existing utilities, relocation of the existing sewer line may be required, however these costs are not included.

8 REFERENCES

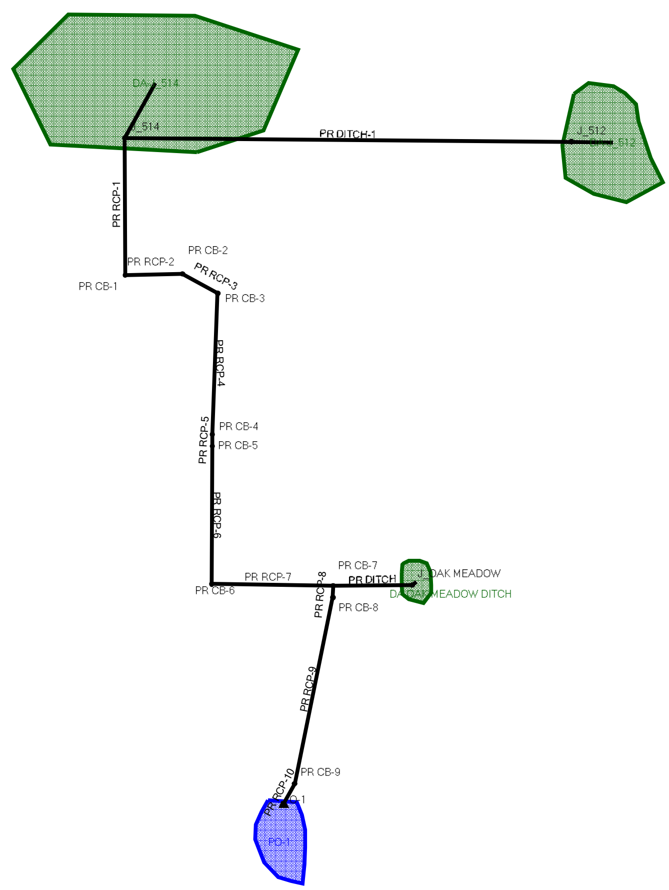
Delaware Department of Transportation (DelDOT), 2008. Road Design Manual Chapter Six: Drainage and Stormwater Management. Available at http://www.deldot.gov/information/pubs_forms/manuals/road_design/pdf/06_drainage_stormwater_mgmt.pdf

Intelisolve Hydraflow Express Version 1.08. Available at http://intelisolve.software.informer.com/

Water Table in the Inland Bays Watershed, Delaware. 2005. Delaware Geological Survey (DGS), accessed February 2015. Available at http://www.dgs.udel.edu/.

NRCS, 2009. U.S. General Soil Survey Geographic Database (SSURGO), accessed November 2014. Available at http://SoilDataMart.nrcs.usda.gov/.

Scenario: 10 Year



CONCEPT DESIGN #5_ OO_22 and OO_28 (OAK MEADOWS)

Project Summary	
Title	
Engineer	
Company	
Date	2/9/2015

Notes

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CONCEPT DESIGN #5_ OO_22 and OO_28 (OAK MEADOWS)

Subsection: Modified Rational Grand Summary

Modified Rational Method

Q = CiA * Units Conversion; Where conversion = 43560 / (12 * 3600)

Frequency (years)	Area ($mile^2$)	Adjusted C Coefficient	Duration (hours)	Intensity (in/h)	Flow (Peak) (ft^3/s)	Flow (Allowable) (ft^3/s)	Volume (inflow) (ac-ft)	Volume (Storage) (ac-ft)
10	0.001	1.000	0.817	2.572	1.66	0.48	0.112	0.081
10	0.020	1.000	5.300	0.699	9.02	2.84	3.950	2.709
10	0.440	1.000	16.600	0.306	86.98	62.47	119.331	33.807

CONCEPT DESIGN #5_ OO_22 and OO_28 (OAK MEADOWS)

Subsection: Master Network Summary

Catchments Summary

Label	Scenario	Return Event (years)	Hydrograph Volume (ac-ft)	Time to Peak (hours)	Peak Flow (ft^3/s)
DA OAK MEADOW DITCH	10 Year	10	0.112	0.500	1.66
DA-J_512	10 Year	10	3.950	0.100	9.02
DA-J_514	10 Year	10	119.331	0.100	86.98

Node Summary

Label	Scenario	Return Event (years)	Hydrograph Volume (ac-ft)	Time to Peak (hours)	Peak Flow (ft^3/s)
J_512	10 Year	10	3.950	0.100	9.02
J_514	10 Year	10	123.281	2.250	96.00
J_OAK MEADOW	10 Year	10	0.112	0.500	1.66
O-1	10 Year	10	128.177	0.350	170.45
PR CB-1	10 Year	10	123.281	2.250	96.00
PR CB-2	10 Year	10	123.281	2.250	96.00
PR CB-3	10 Year	10	123.677	0.400	170.45
PR CB-4	10 Year	10	124.271	0.400	170.45
PR CB-5	10 Year	10	124.663	0.300	170.45
PR CB-6	10 Year	10	125.244	0.300	170.45
PR CB-7	10 Year	10	126.060	0.550	172.11
PR CB-8	10 Year	10	126.769	0.300	170.45
PR CB-9	10 Year	10	127.473	0.350	170.45

CONCEPT DESIGN #5_ OO_22 and OO_28 (OAK MEADOWS)

Subsection: I-D-F Table

Return Event: 10 years

Label: Rainfall Intensity Sussex County

Storm Event: Rainfall Intensity Sussex County - 10 Year

I-D-F Curve

Time (hours)	Intensity (in/h)
0.083	6.760
0.167	5.400
0.250	4.560
0.500	3.300
1.000	2.150
2.000	1.350
3.000	0.990
6.000	0.610
12.000	0.360
24.000	0.220

CONCEPT DESIGN #5_ OO_22 and OO_28 (OAK MEADOWS)

Index

Culvert Report

CONCEPT #5 OO_22-OO_28 PR RCP-1

Invert Elev Dn (ft)	=	9.67
Pipe Length (ft)	=	234.00
Slope (%)	=	0.14
Invert Elev Up (ft)	=	10.00
Rise (in)	=	36.0
Shape	=	Cir
Span (in)	=	36.0
No. Barrels	=	2
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

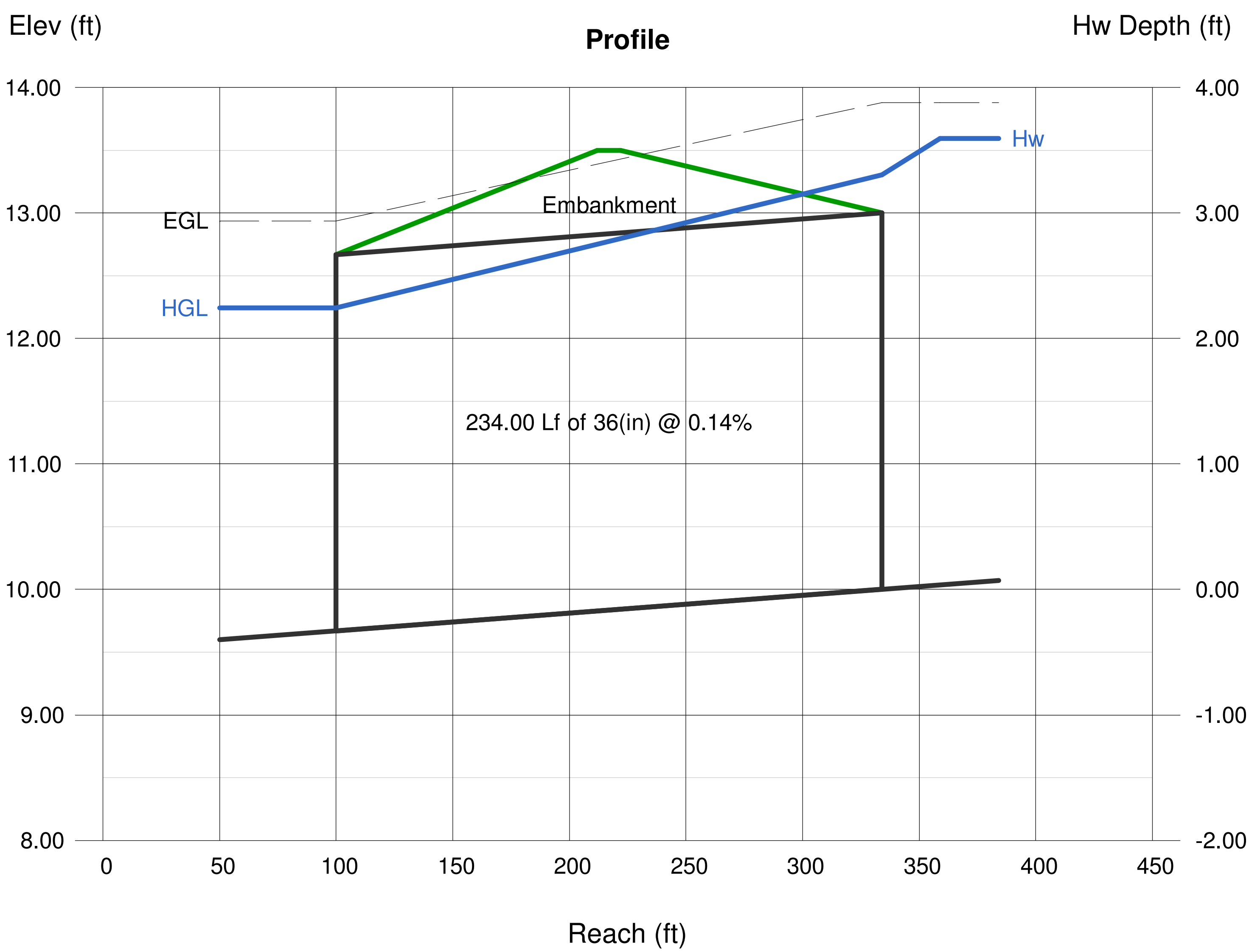
Top Elevation (ft)	=	13.50
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	90.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	87.00
Qpipe (cfs)	=	86.25
Qovertop (cfs)	=	0.75
Veloc Dn (ft/s)	=	6.68
Veloc Up (ft/s)	=	6.10
HGL Dn (ft)	=	12.24
HGL Up (ft)	=	13.30
Hw Elev (ft)	=	13.59
Hw/D (ft)	=	1.20
Flow Regime	=	Outlet Control



Culvert Report

CONCEPT #5 OO_22-OO_28 PR RCP-2

Invert Elev Dn (ft)	=	9.53
Pipe Length (ft)	=	98.36
Slope (%)	=	0.14
Invert Elev Up (ft)	=	9.67
Rise (in)	=	36.0
Shape	=	Cir
Span (in)	=	36.0
No. Barrels	=	2
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

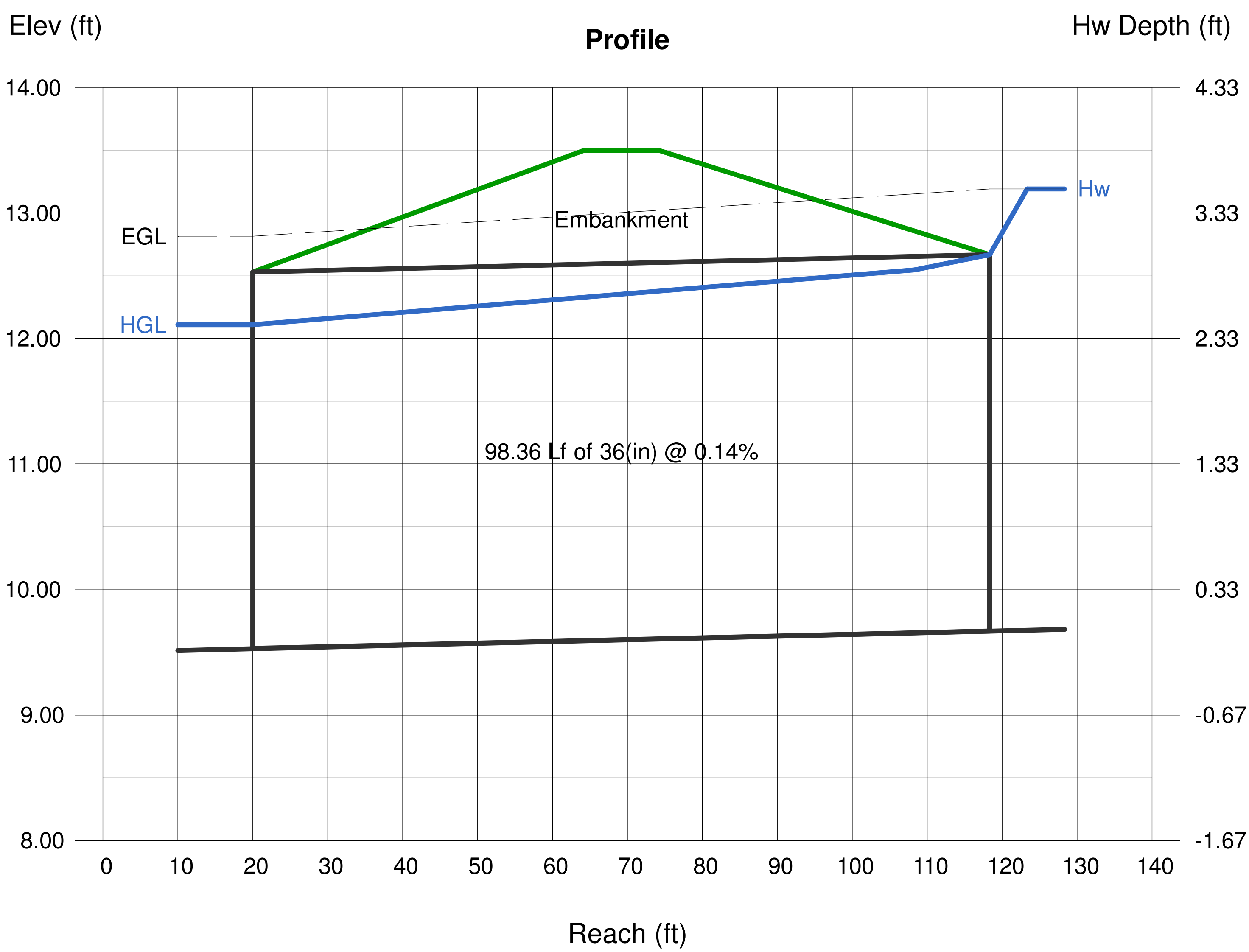
Top Elevation (ft)	=	13.50
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	90.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	87.00
Qpipe (cfs)	=	87.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	6.73
Veloc Up (ft/s)	=	6.19
HGL Dn (ft)	=	12.11
HGL Up (ft)	=	12.60
Hw Elev (ft)	=	13.19
Hw/D (ft)	=	1.17
Flow Regime	=	Inlet Control



Culvert Report

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CONCEPT #5 OO_22-OO_28 PR RCP-3

Invert Elev Dn (ft)	=	9.43
Pipe Length (ft)	=	68.82
Slope (%)	=	0.15
Invert Elev Up (ft)	=	9.53
Rise (in)	=	36.0
Shape	=	Cir
Span (in)	=	36.0
No. Barrels	=	2
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

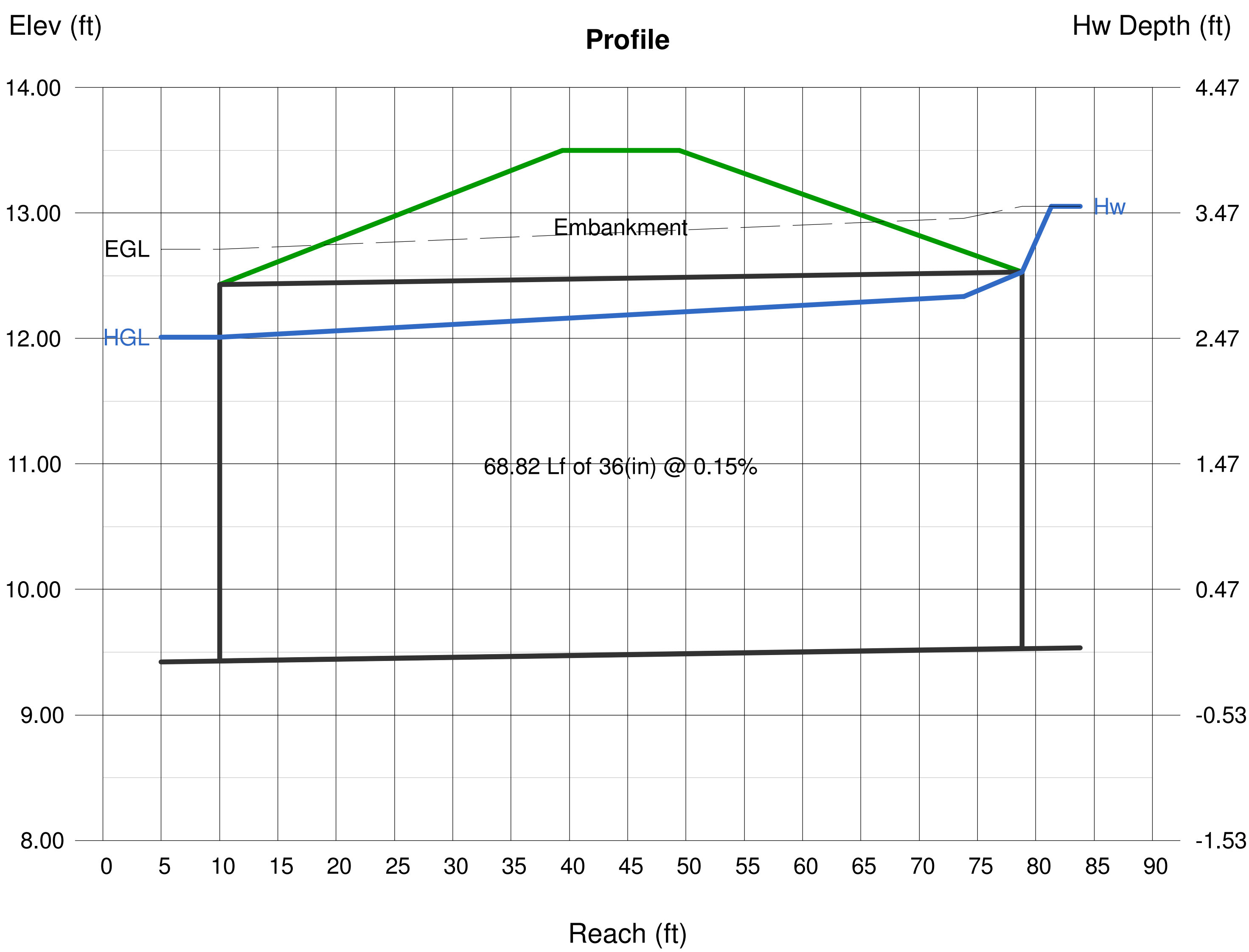
Top Elevation (ft)	=	13.50
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	90.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	87.00
Qpipe (cfs)	=	87.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	6.73
Veloc Up (ft/s)	=	6.30
HGL Dn (ft)	=	12.01
HGL Up (ft)	=	12.36
Hw Elev (ft)	=	13.05
Hw/D (ft)	=	1.17
Flow Regime	=	Inlet Control



Culvert Report

CONCEPT #5 OO_22-OO_28 PR RCP-4

Invert Elev Dn (ft)	=	9.09
Pipe Length (ft)	=	241.37
Slope (%)	=	0.14
Invert Elev Up (ft)	=	9.43
Rise (in)	=	36.0
Shape	=	Cir
Span (in)	=	36.0
No. Barrels	=	2
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

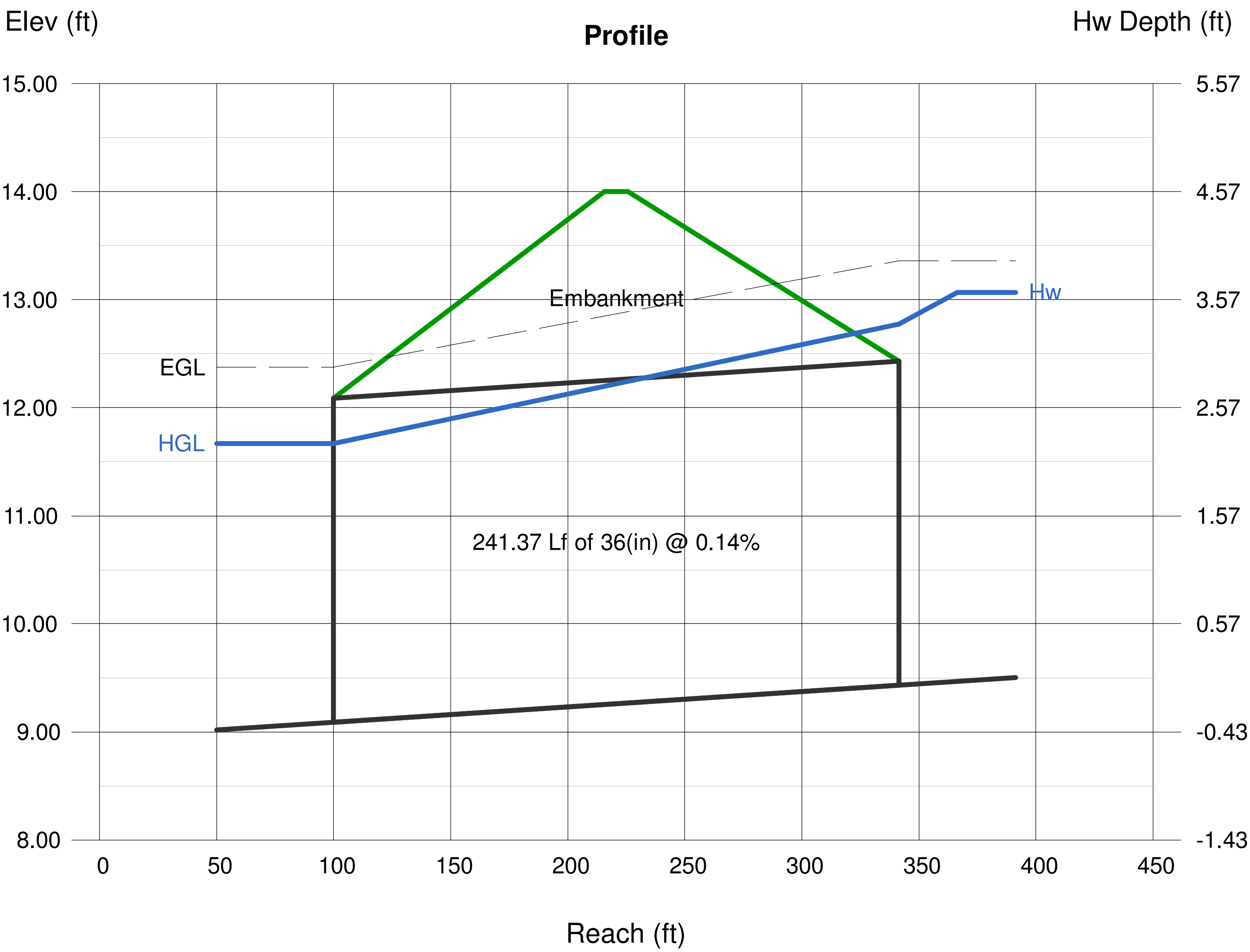
Top Elevation (ft)	=	14.00
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	90.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	87.00
Qpipe (cfs)	=	87.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	6.73
Veloc Up (ft/s)	=	6.15
HGL Dn (ft)	=	11.67
HGL Up (ft)	=	12.77
Hw Elev (ft)	=	13.07
Hw/D (ft)	=	1.21
Flow Regime	=	Outlet Control



Culvert Report

CONCEPT #5 OO_22-OO_28 PR RCP-5

Invert Elev Dn (ft)	=	9.06
Pipe Length (ft)	=	20.15
Slope (%)	=	0.15
Invert Elev Up (ft)	=	9.09
Rise (in)	=	36.0
Shape	=	Cir
Span (in)	=	36.0
No. Barrels	=	2
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

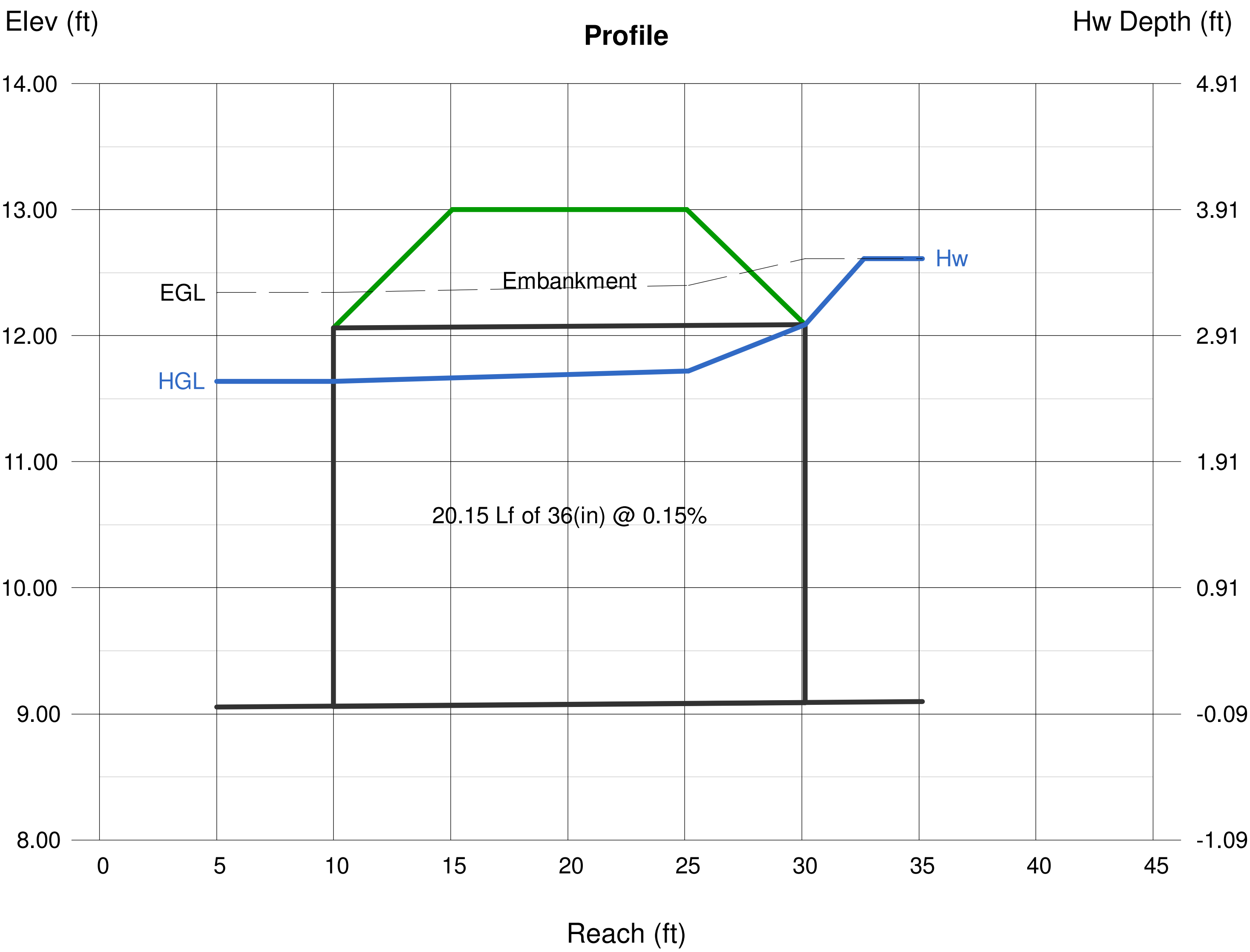
Top Elevation (ft)	=	13.00
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	90.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	87.00
Qpipe (cfs)	=	87.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	6.73
Veloc Up (ft/s)	=	6.56
HGL Dn (ft)	=	11.64
HGL Up (ft)	=	11.75
Hw Elev (ft)	=	12.61
Hw/D (ft)	=	1.17
Flow Regime	=	Inlet Control



Culvert Report

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CONCEPT #5 OO_22-OO_28 PR RCP-6

Invert Elev Dn (ft)	=	8.73
Pipe Length (ft)	=	236.23
Slope (%)	=	0.14
Invert Elev Up (ft)	=	9.06
Rise (in)	=	36.0
Shape	=	Cir
Span (in)	=	36.0
No. Barrels	=	2
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

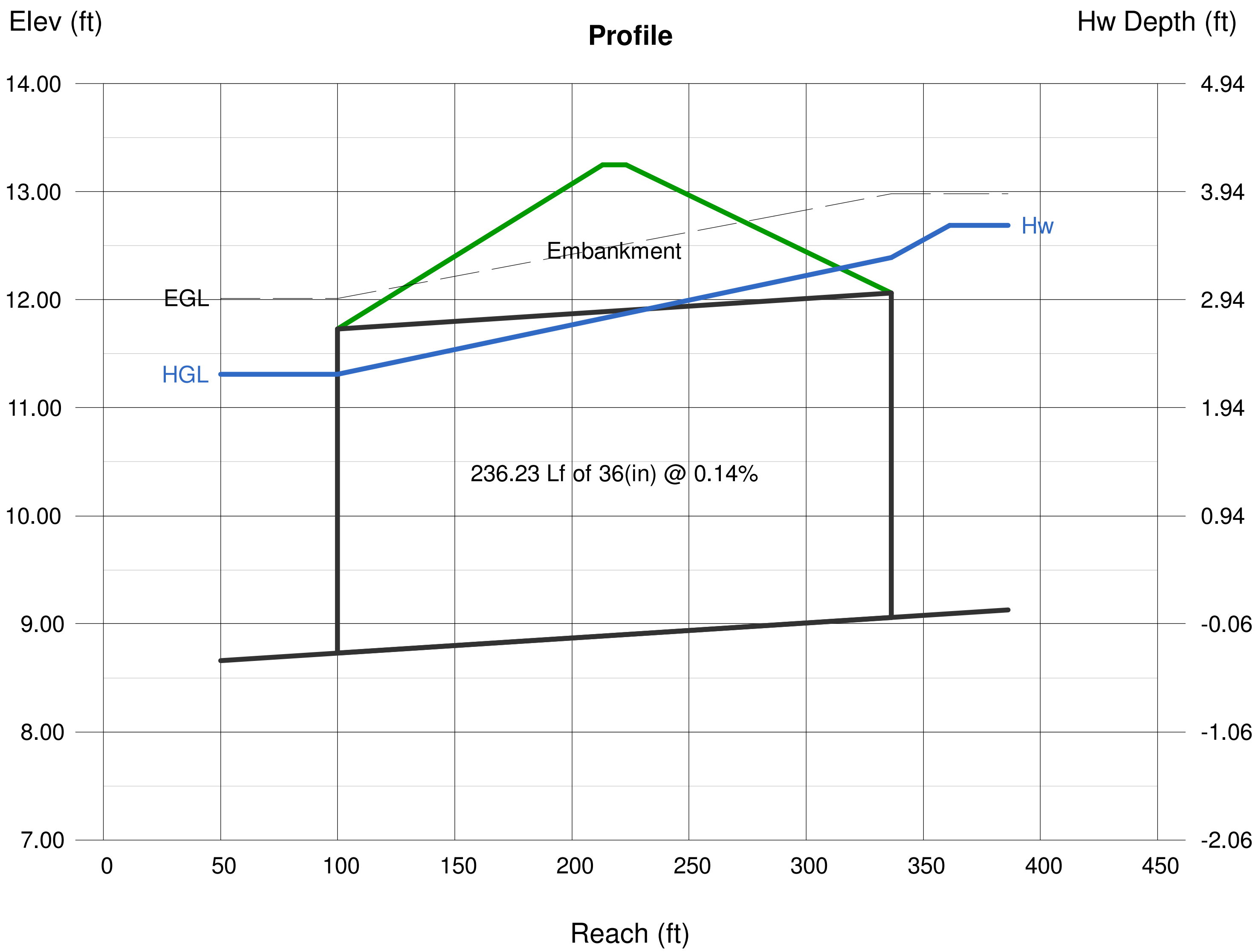
Top Elevation (ft)	=	13.25
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	90.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	87.00
Qpipe (cfs)	=	87.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	6.73
Veloc Up (ft/s)	=	6.15
HGL Dn (ft)	=	11.31
HGL Up (ft)	=	12.39
Hw Elev (ft)	=	12.69
Hw/D (ft)	=	1.21
Flow Regime	=	Outlet Control



Culvert Report

Hydraflow Express by Intelisolve

Friday, Feb 20 2015

CONCEPT #5 OO_22-OO_28 PR RCP-7

Invert Elev Dn (ft)	=	8.44
Pipe Length (ft)	=	207.90
Slope (%)	=	0.14
Invert Elev Up (ft)	=	8.73
Rise (in)	=	36.0
Shape	=	Cir
Span (in)	=	36.0
No. Barrels	=	2
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

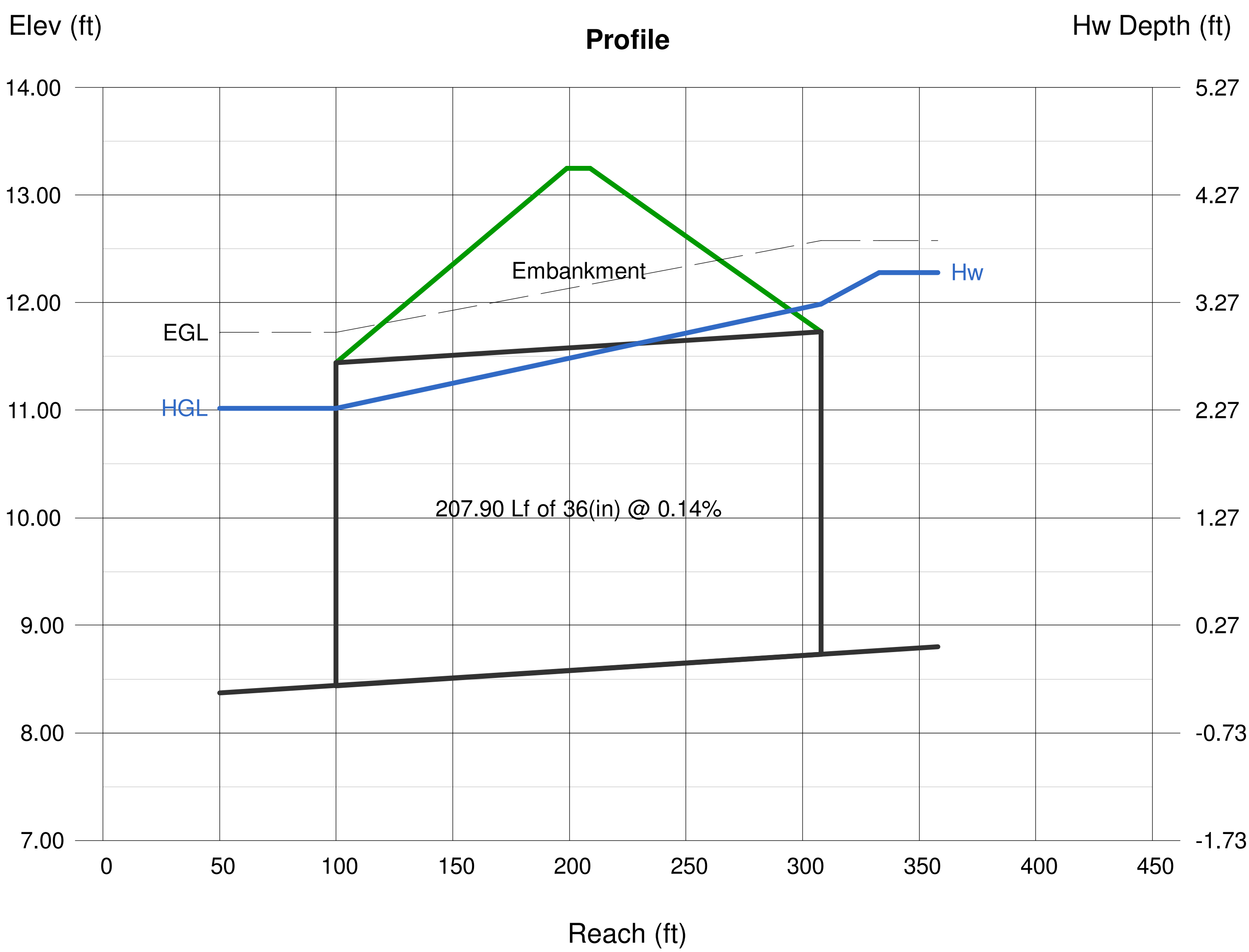
Top Elevation (ft)	=	13.25
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	90.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	87.00
Qpipe (cfs)	=	87.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	6.73
Veloc Up (ft/s)	=	6.15
HGL Dn (ft)	=	11.02
HGL Up (ft)	=	11.99
Hw Elev (ft)	=	12.28
Hw/D (ft)	=	1.18
Flow Regime	=	Outlet Control



Culvert Report

CONCEPT #5 OO_22-OO_28 PR RCP-8

Invert Elev Dn (ft)	=	8.41
Pipe Length (ft)	=	22.91
Slope (%)	=	0.13
Invert Elev Up (ft)	=	8.44
Rise (in)	=	36.0
Shape	=	Cir
Span (in)	=	36.0
No. Barrels	=	2
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

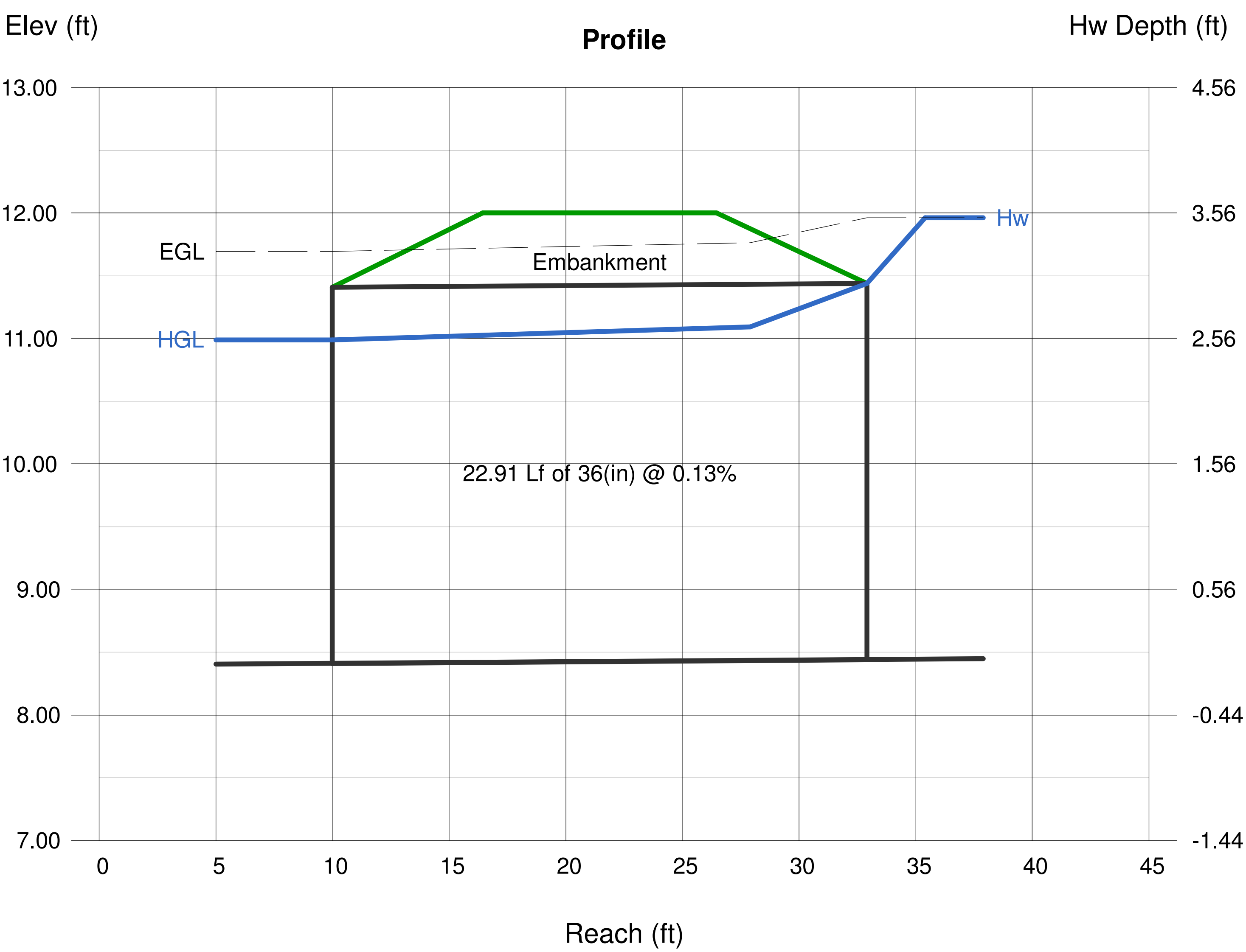
Top Elevation (ft)	=	12.00
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	90.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	87.00
Qpipe (cfs)	=	87.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	6.73
Veloc Up (ft/s)	=	6.53
HGL Dn (ft)	=	10.99
HGL Up (ft)	=	11.12
Hw Elev (ft)	=	11.96
Hw/D (ft)	=	1.17
Flow Regime	=	Inlet Control



Culvert Report

CONCEPT #5 OO_22-OO_28 PR RCP-9

Invert Elev Dn (ft)	=	7.80
Pipe Length (ft)	=	436.90
Slope (%)	=	0.14
Invert Elev Up (ft)	=	8.41
Rise (in)	=	36.0
Shape	=	Cir
Span (in)	=	36.0
No. Barrels	=	2
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

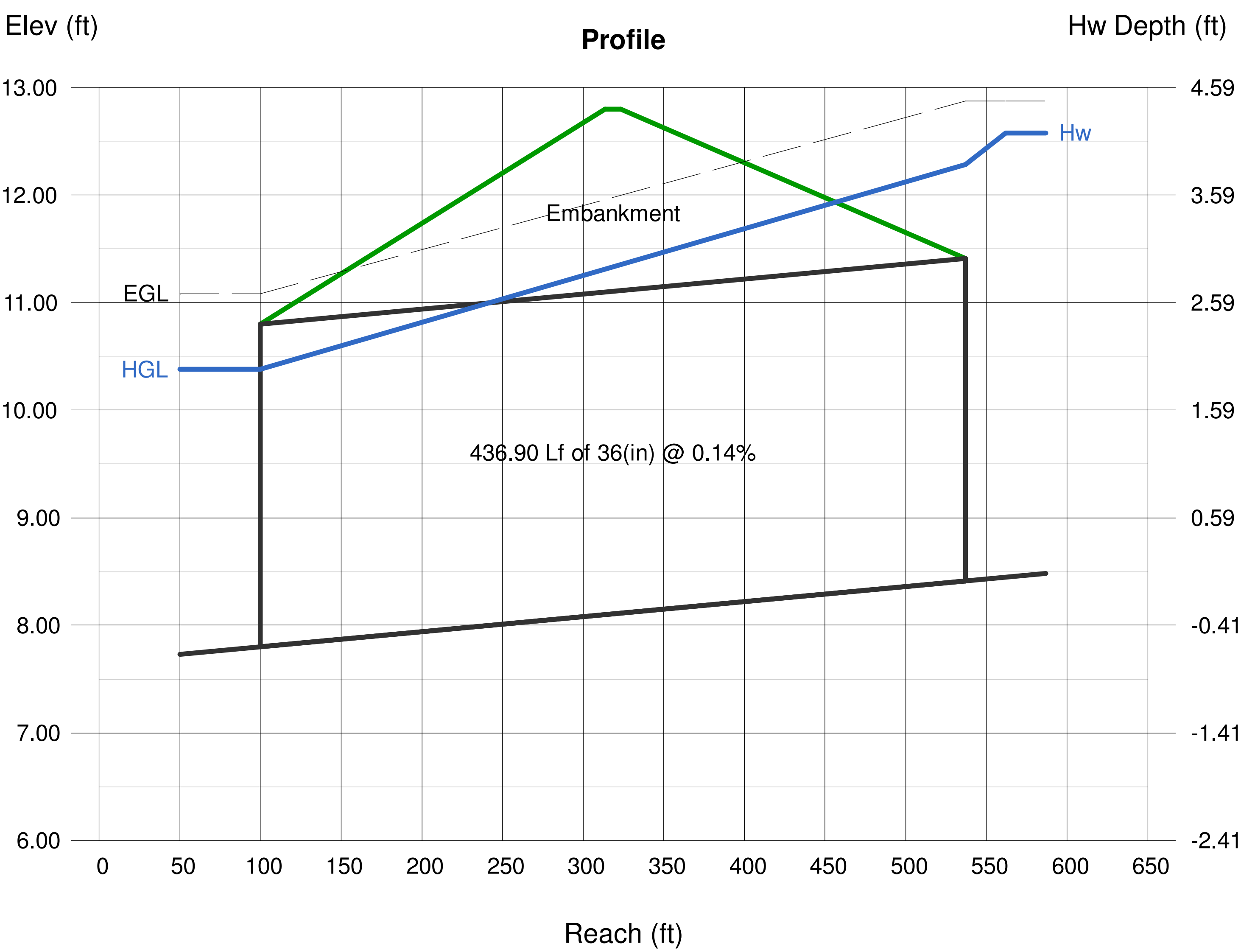
Top Elevation (ft)	=	12.80
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	90.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	87.00
Qpipe (cfs)	=	87.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	6.73
Veloc Up (ft/s)	=	6.15
HGL Dn (ft)	=	10.38
HGL Up (ft)	=	12.28
Hw Elev (ft)	=	12.58
Hw/D (ft)	=	1.39
Flow Regime	=	Outlet Control



Culvert Report

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CONCEPT #5 OO_22-OO_28 PR RCP-10

Invert Elev Dn (ft)	=	7.89
Pipe Length (ft)	=	47.00
Slope (%)	=	0.15
Invert Elev Up (ft)	=	7.96
Rise (in)	=	36.0
Shape	=	Cir
Span (in)	=	36.0
No. Barrels	=	2
n-Value	=	0.013
Inlet Edge	=	Sq Edge
Coeff. K,M,c,Y,k	=	0.0098, 2, 0.0398, 0.67, 0.5

Embankment

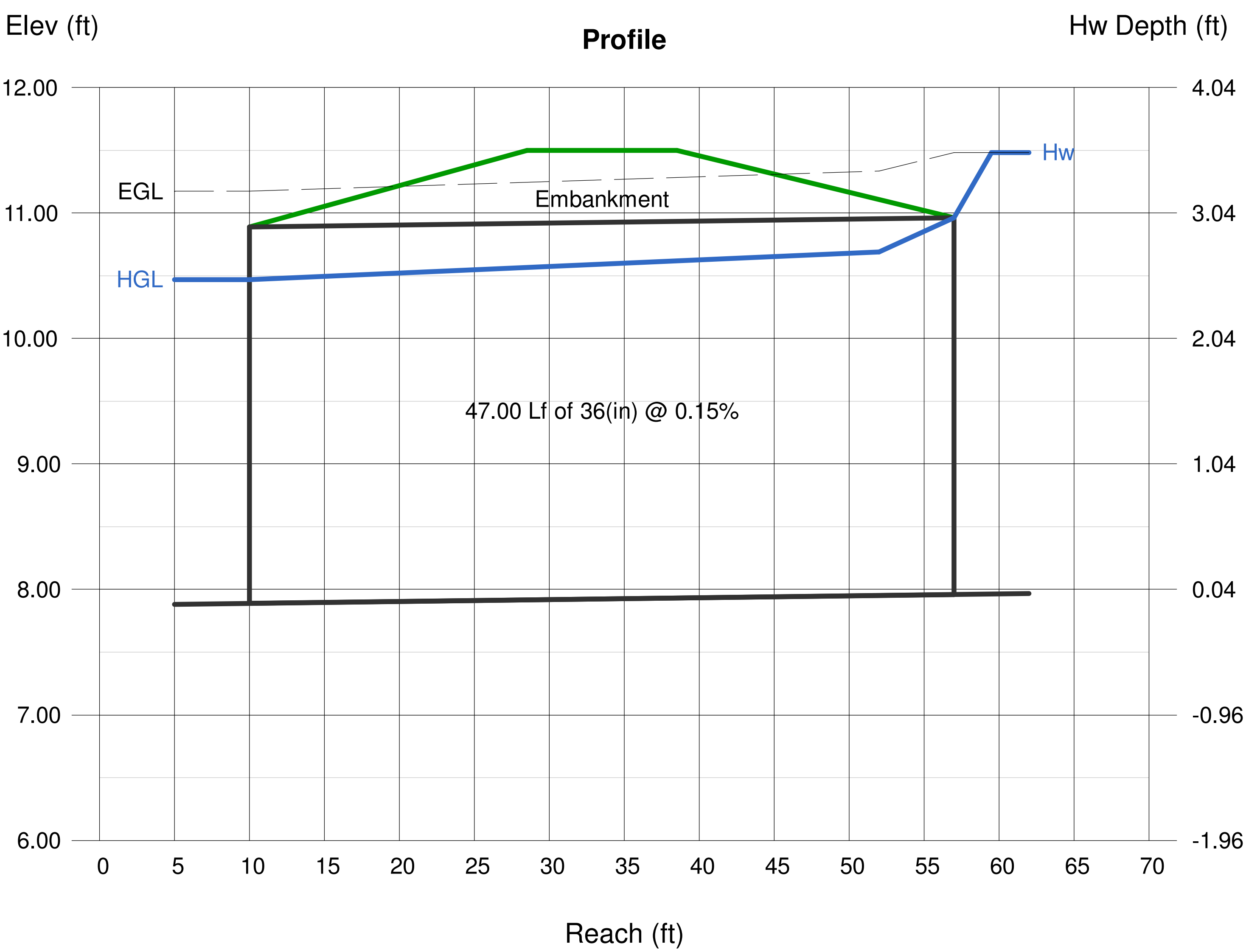
Top Elevation (ft)	=	11.50
Top Width (ft)	=	10.00
Crest Width (ft)	=	10.00

Calculations

Qmin (cfs)	=	0.00
Qmax (cfs)	=	90.00
Tailwater Elev (ft)	=	(dc+D)/2

Highlighted

Qtotal (cfs)	=	87.00
Qpipe (cfs)	=	87.00
Qovertop (cfs)	=	0.00
Veloc Dn (ft/s)	=	6.73
Veloc Up (ft/s)	=	6.40
HGL Dn (ft)	=	10.47
HGL Up (ft)	=	10.71
Hw Elev (ft)	=	11.48
Hw/D (ft)	=	1.17
Flow Regime	=	Inlet Control



Channel Report

Hydraflow Express by Intelisolve — Friday, Feb 13 2015

OO_22 PR DITCH

Trapezoidal
Botom Width (ft) = 2.00
Side Slopes (z:1) = 2.00, 2.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 1.00
Slope (%) = 0.50
N-Value = 0.030

Calculations
Compute by: Known Depth
Known Depth (ft) = 1.50

Highlighted
Depth (ft) = 1.50
Q (cfs) = 23.78
Area (sqft) = 7.50
Velocity (ft/s) = 3.17
Wetted Perim (ft) = 8.71
Crit Depth, Yc (ft) = 1.14
Top Width (ft) = 8.00
EGL (ft) = 1.66